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Quality Control and Evaluation of Steel Fiber Reinforced Concrete

Study SD87-02 Final Report

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16. Abstract

This report presents the construction and performance evaluation of steel fiber reinforced concrete (SFRC) pavements constructed in Rapid City, SD, in the summer of 1988 (Haines Avenue) and in the winter of 1990 (Sheridan Lake Road). The primary objective of this evaluation was to compare the performance of SFRC pavement with that of non-reinforced concrete pavement.

Using the selected mixture proportions with 0.5 volume percent of steel fibers, the Haines Avenue pavement was constructed with hooked end steel fibers and the Sheridan Lake Road pavement was constructed with corrugated steel fibers. Large number of test specimens were made from the field concretes and their basic properties and performance characteristics were evaluated. SFRC exhibited superior performance characteristics - it had higher flexural strength, considerably higher fatigue and impact strength, higher toughness, higher energy absorbing capacity and better post-crack load carrying capacity. Concrete cores taken from the pavements had shown that SFRC was well compacted, the fiber distribution was uniform throughout the depth, and the in-place compressive strength exceeded that of the design strength.

The performance of the pavements was evaluated for a period of 5 years by conducting detailed inspection at every 3 months. The results of this investigation have shown that SFRC pavements are performing much better than the thicker non-reinforced concrete pavements.

Recommendations are made for the implementation of the findings of this experimental project.

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EXECUTIVE SUMMARY

INTRODUCTION & PROBLEM STATEMENT

Earlier research had established that the addition of steel fibers in concrete improves the static flexural strength, flexural fatigue strength, impact strength, shock resistance, ductility and fatigue toughness. Use of steel fiber reinforced concrete (SFRC) allows a substantial reduction in concrete thickness for the given loads when compared to conventionally designed pavements. Because of small crack openings, durability is also improved by preventing the ingress of deteriorating substances. Joint spacing can be considerably increased, thus reducing the number of joints needing future maintenance. Impact to heavy wheel loads is greatly improved.

The important design parameters in many applications especially in pavements and bridge deck overlay, are the flexural fatigue strength and the endurance limit on the basis of which the pavements are designed. By the addition of fibers to the concrete, the flexural strength both in the static and dynamic loading is improved to a great extent. The fatigue endurance limit of plain concrete is about 50 to 55% of its static flexural strength. With the proper design of fiber reinforced concrete, a comparatively higher endurance limit can be achieved. Thus concrete pavements and decks could last for a much longer time period with more load carrying capacity. Therefore it could be concluded that SFRC is a suitable material for the construction of highway pavements.

PROJECT WORK PHASES

The Rapid City Engineering Department, initiated by Lawrence Kostaneski, Project Engineer, had constructed a section of the Haines Avenue pavement with SFRC during the normal construction operations in June 1988. This was a Federal Aid Urban Systems Reconstruction Project, 2420 ft (737.62 m) long, 48 ft (14.63 m) wide, designed with 7.5 inch (190.5 mm) thick, non-reinforced PCC pavement.

The SFRC pavement consisted of two sections 90 ft (27.43m) and 75 ft (22.86m) long with 6 inch (152.4mm) and 5 inch (127mm) thicknesses, with three transition sections of lengths 15 ft (4.57m) each. Transverse joints were placed at 15 ft (4.57m) center and longitudinal joints were placed at 12 ft (3.66m) center.

The initial testing and evaluation, and the periodic inspections had shown that after 18 months of service, the condition of the SFRC pavement was highly satisfactory. Encouraged by the success, Rapid City Engineers constructed a section of the Sheridan Lake Road pavement with SFRC. In order to minimize the cost, a less expensive, different type of steel fiber, was used for this new pavement. Appropriate mix proportions, quality control during construction and performance testing and periodic inspection for a period of 5 years after construction were provided by Ramakrishnan.

MATERIALS AND MIXTURE PROPORTIONS

The hooked end fibers used for the construction of Haines Avenue Pavement had deformed ends and were glued together in bundles. Corrugated steel fibers were used in the construction of Sheridan Lake Road Pavement. Type I/II portland cement was used for all the construction. The fine aggregate consisted of natural sand and the coarse aggregate consisted of crushed limestone, 1½ in. (38mm) maximum size.

Based on an extensive research experience and in consultation with the fiber supplier, a mix proportion was selected to give a compressive strength of 6000 psi (41.37 MPa) with a cement content of 525 lbs/cu.yd (312.11kg/cu.m). For economical reasons, the lowest possible quantity of steel that will be effective in enhancing the concrete properties to the specified level was selected as 0.5 percent by volume of concrete. A laboratory investigation of the selected mixture proportions by trial mixes had confirmed its suitability.

QUALITY CONTROL AND PERFORMANCE TESTING

The SFRC pavement at Haines Avenue was constructed on June 1 (north bound lanes) and June 7, (south bound lanes). The Sheridan Lake Road pavement was constructed on October 12, 1990 (south bound lanes) and November 15, 1990 (north bound lanes). During the construction the following quality control tests were conducted on fresh and hardened concrete by taking large number of samples from SFRC placed in the pavement.

Fresh Concrete : Slump Test, Inverted Cone Test, Unit Weight, Air Content, actual fiber content and fiber distribution. The temperature of the concrete, the ambient temperature, humidity and the wind velocity were recorded. The finishability of the concrete was observed and recorded.

Hardened Concrete : Two size beams, 6"x6"x21" (152.4x152.4x533.4mm) and 4"x4"x14" (101.6x101.6x355.6mm) and Impact specimens 2.5"x6" (63.5x152.4mm) and cylinders 6"x12" (152.4x304.8mm) were made during the construction and they were tested for the following properties: compressive strength at 1, 3, 7, 14, and 28 days; flexure tests at 7 and 28 days; toughness index, impact test, modulus of elasticity, pulse velocity; load deflection curves including the first crack strength, and flexural fatigue strength.

These tests were conducted according to ASTM and ACI recommended test procedures.

The fiber reinforced concrete was relatively easy to work with and to place. No balling or tangling of fibers occured during mixing. It had less bleeding compared to plain concrete and had good finishability. Due to the addition of fibers, the ductility and energy absorption capacity were greatly improved. There was an increase (21%) in the static flexural strength and (80%) in flexural fatigue strength as compared with normal concrete without fibers. The impact resistance was also improved considerably. In general, the results indicated that the concrete in place was satisfactory.

CONSTRUCTION

Preconstruction meetings were held during both phases of construction. The concrete was supplied by a central ready mix plant. Only the conventional equipment was used in mixing, transporting, placing, consolidating and finishing the SFRC pavements. No new or additional equipment was needed. A concrete paver (GOMACO, IDA Groove, Iowa) was used operating at a vibrator frequency of 8000 vibrations per minute. A rollerbug was used for finishing. After the rollerbug use, the surface was finished with long wooden floats. The pavement was then broom finished with a wide nylon brush.

Finally the finished surface was sprayed with the curing compound. The joints were saw-cut as early as permissible.

When the north bound lanes of Haines Avenue were constructed, the weather was ideal for concreting (62°F and a relative humidity of 55%), but when the south bound lanes were built, it was one of the most unfavorable days for concreting (with an ambient temperature of 105°F and a relative humidity of 10%). These two conditions had provided us an opportunity to evaluate SFRC performance in two different weather conditions. There was a 55°F variation between the hot day and the cool night temperature during the curing period.

MODIFICATIONS MADE FOR SFRC PAVING

The modifications necessary for proper paving with SFRC were (a) fly ash was added to the mixture as an alternative to the addition of a superplasticizer in order to obtain an acceptable workability (b) adding the fibers to the coarse aggregate and mixing to disperse the fibers prior to adding the sand, cement and water to prevent balling; (c) adjusting the vibration frequency of the spud vibrators to improve consolidation of the SFRC; (d) adjusting the tension on the astroturf drag to prevent pulling of the fibers onto the surface of the pavement; (e) requiring the broom finish to be applied in the same direction as the paving operation to minimize the exposure of steel fibers; (f) noting that tining is not recommended with steel fibers due to raveling.

In the case of SFRC, stricter quality control and additional care was needed during mixing and placing. Additional cure time was needed for SFRC as compared to plain concrete before joints could be saw-cut. Otherwise, fibers tended to ravel joints when contacting the blades.

INSPECTION

Periodic field inspections were made to observe the crack formations, fiber protrusions, surface conditions, and any other distress in the pavement. Observations were made both on the steel fiber reinforced concrete and the non-reinforced concrete sections in order to have a comparative evaluation. During the first inspection it was found that only one crack had appeared in the fiber reinforced section while a number of cracks had appeared in the non-reinforced concrete section. The lengths of the cracks varied from 5 to 60 inches (127 to 1524mm) and the width of the crack varied from 0.008 to 0.015 inches (0.20 to 0.40mm). Mostly the cracks had appeared near the intersection of the longitudinal and transverse joints and they were in the middle of the pavement. Most of the cracks were thin hair line cracks and they formed very close and parallel to the transverse saw-cut joints. These cracks formed on that part of the pavement which was constructed on June 7, 1988 when the weather was most unfavorable. This particular portion of the slab was cast during the hottest time of the day.

Based on the comparison of the south bound and north bound lanes of the pavements, it is concluded that these cracks were the result of weather conditions, and not due to any inherent material defects. These cracks could have been avoided, or eliminated to a great extent by adopting appropriate precautionary measures for hot weather concreting.

The inspections had revealed that there were no fiber protrusions, or fibers sticking out at the surface. There were few (less than one fiber per square yard) fibers exposed at the surface ; they were lying flat. This is a very good achievement, much better than the anticipated result. Further periodic inspections of the pavement at 3 month intervals were made upto 30 April, 1993.

In the SFRC section there were no fiber protrusions or loose fibers on the pavement. However there were some fibers exposed, still attached to the pavement, lying flat. Near the surface where there was heavy wheel traffic, a careful inspection revealed that there were some steel fibers exposed and fully bonded to the matrix. They were bright and shiny indicating that they were undergoing the same wear as the coarse aggregate pieces. They easily merged with the concrete surface and it was not easy to distinguish them from the aggregate pieces. These fibers do not seem to cause any problems; they do not seem to be any nuisance or present a hazard. The normal wearing and spalling seen at the joints were the same for both the reinforced and unreinforced parts of the pavements. In general, it can be stated that after 60 months of service, the condition of SFRC pavement was satisfactory.

There was no recognizable difference in the skid resistance of the plain concrete and SFRC pavements. After 3 years the skid resistance was still satisfactory.

CORE TEST RESULTS

Cores were taken to verify the thickness of the steel fiber reinforced concrete pavement, to check for the consolidation and fiber distribution, and to determine the compressive strengths. The measured dimensions were never less than 5 and 6 inch (127 and 152.4mm) thick at specified locations. The observed variations in the depth are within permissible limits.

An inspection of the cores had shown that the consolidation of the concrete was good. There were no large honeycombs or voids. The fiber distribution across the depth seemed satisfactory.

COST COMPARISON

The cost comparison is based on the bid prices of successful contractors for the Haines Avenue and Sheridan Lake Road pavement constructions. The unit prices (per square yard) for SFRC were the same for both Haines Avenue and Sheridan Lake Road pavements even though the market price of the corrugated steel fiber was considearbly less than that of the hooked-end fibers. The unit cost of 5 inch (127mm) thick and 6 inch (152mm) thick SFRC were $25.00 anf $28.00 respectively whereas the cost of 8 inch (203mm) thick non-reinforced pavement was $20.00. It must be recognized that this comparison is based on the misleading high costs associated with experimental construction.

UTILIZATION OF RESULTS

The results of this investigation will be not only useful to the SDDOT, but also to all other states. The implementation of the findings has excellent potential for economic as well as technical advantages in the field of pavement construction. The benefit derived to the nation will be substantial.

CONCLUSIONS & RECOMMENDATIONS

Some of the problems encountered with SFRC were overcome without undue extra cost or modifications of standard paving practice. With a 0.5 volume percent of steel fibers, SFRC provided a better performance specially, when less than perfect conditions existed in the field. Both SFRC pavement sections with reduced thicknesses (20% and 33.3%) performed equally well compared to the non-reinforced concrete pavements.

Therefore it is recommended that steel fiber reinforced concrete (SFRC) could be used for the construction of pavements. This will enhance the performance and structural efficiency of the pavement with a potential for longer life. FRC pavements could be used in all highways, urban, or rural or interstate highways, either with high density or low-density traffic. When FRC is used for the construction of pavements, the thickness can be reduced without compromising its strength or performance. It is recommended that a 20 percent reduction in thickness can be made to reduce the total cost of the pavement. This is a conservative recommendation considering the fact that in the experimental project, the SFRC pavement with 33.3 percent reduced thickness performed well. However this reduction should depend on the fiber content and the type and efficiency of the fibers used. It is possible to achieve further reduction with higher fiber content or with more efficient fibers. When steel fibers are used, it is recommended that a minimum of 0.5 volume percent of fibers should be used in the case of efficient fibers. Higher quantities of fibers should be used in the case of less efficient fibers. It is recommended that the basic properties and the performance characteristics, particularly the performance when subjected to fatigue loading, should be evaluated for the proposed or selected mixture proportion for the FRC. It is recommended that strict quality control measures should be followed when FRC is used in pavement construction.

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INTRODUCTION

Random oriented steel fiber reinforced concrete is a promising constructing material for highway pavements. To increase the fracture resistance of cementitious materials, steel fibers are frequently added, thus forming a composite material. Although the concept of reinforcing brittle material with fiber is quite old, the recent interest in reinforcing portland cement based materials with randomly distributed fibers was spurred by pioneering research on steel fiber reinforced concrete conducted in the United States in the 1960's.

Concrete fiber composites have been found more economical for the use in highway and airport pavements, bridge deck overlay, curtain walls, sewer pipes, cavitation/erosion resistance structures, security structures, tunnel linings, rock slope stabilization, earth-quake resistance structures (like MX missile silos) etc. The behavioral efficiency of fiber reinforced concrete is far superior to that of plain concrete and many other construction materials of equal cost (1).

An extensive research to develop tough, high-strength, high density, durable concrete for the construction of highway pavements and bridge decks, through the use of steel fibers and superplasticizers had been conducted earlier which was sponsored by the U.S. Department of Transportation (3). This work has established that the addition of steel fibers in concrete improves the static flexural strength, flexural fatigue strength, impact strength, shock resistance, ductility and fatigue toughness (3). Use of steel fiber reinforced concrete allows a substantial reduction in concrete thickness (40 to 50 percent) for the given loads when compared to conventionally designed pavements. Because of small crack openings, ductility is also improved by preventing the ingress of deteriorating substances. Joint spacing can be considerably increased, thus reducing the number of joints needing future maintenance. Impact to heavy wheel loads is greatly improved (3).

Steel fiber reinforced concrete(SFRC) has been used throughout the world in pavements and bridge decks. Numerous applications of SFRC in pavements have been given by George Hoff with some performance information and an extensive bibliography(1). Moussa Bagate, Frank Mccullough and David Fowler have presented the details about construction and performance of an experimental thin-bond overlay pavement in Houston. Recently the Quebec Department of Transportation has constructed SFRC pavements.

Practical construction problems were experienced in early field work concerning production rate, mix design, workability, addition of fibers to the mix, finishing, pavements developing curls, problem at joints, cracking and potentially hazardous surface fibers. However, the industry now knows how to deal with these construction problems and to successfully eliminate them. Based on the extensive research work done by many researchers (2) and a thorough investigation of the recently constructed pavements, it could be concluded that steel fiber reinforced concrete is an ideally suitable material for the construction of highway pavements.

PROJECT

Recognizing the advantages of using Steel Fiber Reinforced Concrete for highway pavements, Rapid City Engineering Department, initiated by Lawrence Kostaneski, Project Engineer, had constructed a section of the Haines Avenue pavement with SFRC during the normal construction operations in June 1988. This was a Federal Aid Urban Systems Reconstruction Project, 2420 ft (737.62 m) long, 48 ft (14.63 m) wide, designed with 7.5 inch (190.5 mm) thick, non-reinforced PCC pavement.

The SFRC pavement consisted of two sections 90 ft (27.43m) and 75 ft (22.86m) long with 6 inch (152.4mm) and 5 inch (127mm) thicknesses, with three transition sections of lengths 15 ft (4.57m) each. Transverse joints were placed at 15 ft (4.57m) center and longitudinal joints were placed at 12 ft (3.66m) center.

The initial testing and evaluation, and the periodic inspections had shown that after 18 months of service, the condition of the SFRC pavement was highly satisfactory and the

SFRC pavement had been an unqualified success. Encouraged by the success, Rapid City Engineers wanted to construct a section of the proposed Sheridan Lake Road pavement with SFRC. In order to minimize the cost, a less expensive, different type of steel fiber, which was expected to give similar performance as the previously used more expensive fiber, was used for this new pavement. Therefore the same work, providing appropriate mix proportions, quality control during construction and performance testing and periodic inspection for a period of 3 years after construction was repeated for this pavement.

OBJECTIVES

The major objectives of this project were:

1. To select appropriate mix proportions to obtain the optimum performance efficiency and specified properties consistent with economical cost.
2. To perform quality control and performance testing of fresh and hardened concrete.
3. To periodically conduct a condition survey of SFRC pavement and to evaluate its performance.
4. To compare the performance of SFRC pavement against standard portland cement concrete non-reinforced pavement.

SIGNIFICANCE OF THIS INVESTIGATION

Fiber reinforced concrete is becoming more and more common material for the use in airport and highway pavement construction because of its improved performance and economy.

The important design parameters in many applications especially in pavements and bridge deck overlay, are the flexural fatigue strength and the endurance limit on the basis of which the pavements are designed. By the addition of fibers to the concrete, the flexural strength both in the static and dynamic loading is improved to a great extent (1).

The fatigue endurance limit of plain concrete is about 50 to 55% of its static flexural strength. With the proper design of fiber reinforced concrete a comparatively higher endurance limit can be achieved. Thus the concrete pavement and the decks could last for a much longer time period with more load carrying capacity (3).

MATERIALS, MIXES, AND TEST SPECIMENS

Materials

The hooked end fibers used for the construction of Haines Avenue Pavement had deformed ends and were glued together in bundles. These fibers were made from low carbon steel wires with one standard size having a nominal length of 2.36 in. (60 mm) and a diameter of 0.03 in. (0.8 mm) with an aspect ratio of 75. Corrugated steel fibers were used in the construction of Sheridan Lake Road Pavement. These fibers were 2 inch (50.9mm) long conforming to ASTM A820, Type I. The equivalent diameter of the fibers was 0.03 to 0.05 inch (0.76mm to 1.27mm) with an aspect ratio of 40 to 65. Type I/II portland cement was used for all the construction. The fine aggregate consisted of natural sand obtained from Birdsall Sand and Gravel Co., Rapid City, SD. The coarse aggregate consisted of crushed limestone, 1½ in. (38mm) maximum size obtained from Pete Lein Sons quarry in Rapid City, SD.

Mixes

The concrete mix was basically divided into two types, trial mix and the field mix. The trial mix was supplied by the concrete contractor (central ready mix plant), and it was mixed using the same materials and the same procedure that were used for the actual field concrete.

Depending upon the water cement ratio and some other factors influencing the mix, the trial mix and the field mix had been further divided into SD-I, SD-II, SDP-I, SDP-II respectively. Table 1 shows the mix designation. A few plain concrete specimens were also cast from the field mix in order to compare the results with those of fiber reinforced concrete, and they are identified as SDP-IP. Table 2 gives the basic mix proportion used for the field mix (SDP).

WORK PERFORMED

Selection of appropriate mixture proportions

Based on an extensive research experience of the principal investigator, his consulting experience in regard to SFRC, and in consultation with the fiber supplier (Pete Tatnall), a mix proportion was selected to give a compressive strength of 6000 psi (41.37 MPa) with a cement content of 525 lbs/cu.yd (312.11kg/cu.m). A trial mix was made using the same materials and the same procedure that were going to be used for actual field concrete. The concrete supplier for the project (Central Mix, Rapid City) supplied the concrete to the SDSM&T lab. and the staff of Rapid City Engineering Department and the concrete contractor were also invited to participate in the evaluation of fresh concrete properties like finishability and workability. Specimens were made and tested for the determination of all hardened concrete properties. Results are included in the Supplementary Report 1 (5). Based on the test results and in consultation with Rapid City Engineers and the concrete contractor, the final mix proportions were adopted.

Preconstruction Meeting for Haines Avenue Pavement Construction

With a request from the principal investigator three preconstruction meetings were arranged by the city engineers and were held on March 3, May 6, May 20, 1988. People in attendance at the first meeting were Larry Kostaneski, Rich Wells, Rodell Grosz, Dallas Thomas, Lyle Quenzer, Jerry Havens (Rapid City Engineering Department), Terry Larson (Heavy Constructors), Bill Whitney (Stanely Johnson), Ramakrishnan (SDSM&T), Pete Tatnall (fiber supplier). Trial mix details, construction procedures and control tests were discussed. The second meeting (on May 6) was attended by the following persons : Mike

Vidal (Birdsall Ready Mix), Rama (SDSM&T), Bill Whitney (Concrete Contractor), Steve Horman (Heavy Constructors), and Rodell Grosz (Rapid City Engineering Department). Schedule of trial mix, and SFRC construction dates were agreed.

The third meeting (on May 20) was attended by Mike Vidal, Ramakrishnan and Rodell Grosz. Test results of the trial mix and construction dates were discussed.

Preconstruction Meeting for Sheridan Lake Road Pavement Construction.

A preconstruction meeting was held on October 8, 1990 at the South Dakota School of Mines and Technology, Civil Engineering Department Conference Room. Representatives from the contractor, concrete suppliers, Rapid City Engineering Department and the South Dakota Department of Transportation were present. The following personnel attended the meeting.

1. Dr. V. Ramakrishnan, monitor of the project
   South Dakota School of Mines & Technology
2. Mr. Todd Seaman, South Dakota Department of Transportation
3. Mr. Klare Schroeder, Rapid City Engineering Department
4. Mr. Larry Van Osdel, Sweetman Construction Company Rapid City, SD
5. Mr. Jerry Schnabel, Hills Materials Company, Rapid City, SD
6. Mr. Bob Goodrich, Central Mix, Rapid City, SD

The objectives of this experimental steel fiber reinforced concrete pavement (SFRC) construction on Sheridan Lake Road, Rapid City, were stated. The advantages of SFRC pavements were explained. Since all persons present were not directly involved with the previous SFRC pavement construction at Haines Avenue, the procedures used for the above construction were described. It was decided to follow the same procedures as far as possible.

The same mixture proprotions were used except the steel fibers used were corrugated, 2-inch (51mm) long fibers confirming to ASTM A820, Type 1. These fibers were cheaper than the hooked-end fibers used in the Haines Avenue construction. However, the same quantity of fibers 66 lbs/cu.yd (39.24kg/cu.m) was used in this project. The objective was to determine whether the same performance efficiency or at least the specified performance can be achieved with a lower unit cost for concrete.

All present promised full cooperation with the monitor and the procedures to be followed were decided. The monitor explained his role and functions in this project. The state engineer and the contractor presented a tentative schedule of SFRC pavement construction.

Quality Control and Performance Testing

The SFRC pavement at Haines Avenue was constructed on June 1, (north bound lanes) and June 7, (south bound lanes). During the construction the following quality control tests were conducted on fresh and hardened concrete by taking samples from SFRC placed in the pavement.

Fresh Concrete : Slump Test, Inverted Cone Test, Unit Weight, Air Content, actual fiber content and fiber distribution. The temperature of the concrete, the ambient temperature, humidity and the wind velocity were recorded. The finishability of the concrete was observed and recorded.

Hardened Concrete : Two size beams, 6"x6"x21" (152.4x152.4x533.4mm) and 4"x4"x14" (101.6x101.6x355.6mm) and Impact specimens 2.5"x6" (63.5x152.4mm) and cylinders 6"x12" (152.4x304.8mm) were made during the construction and they were tested for the following properties: compressive strength at 1, 3, 7, 14, and 28 days; flexure tests at 7 and 28 days; toughness index, impact test, modulus of elasticity, pulse velocity; load deflection curves including the first crack strength, and flexural fatigue strength.

Whenever applicable the above tests were conducted according to ASTM and ACI recommended test procedures.

The fiber reinforced concrete was relatively easy to work with and easy to place. No balling or tangling of fibers occured during mixing. SFRC had less bleeding compared to plain concrete and good finishability. Due to the addition of fibers, the ductility and energy absorption capacity were greatly improved. There was an increase (21%) in the static flexural strength and (80%) in flexural fatigue strength as compared with normal concrete without fibers. The impact resistance was also improved considerably. In general, the results indicated that the concrete in place was satisfactory.

A comparison of the properties of the control concrete with SFRC is given in Table 2A.

CONSTRUCTION OF HAINES AVENUE PAVEMENT

In the actual mix, using the selected mix proportions, the fiber concrete was mixed in the central mix plant and brought to the field in concrete trucks. The quantity of fiber mixed was 66 lbs/cu.yd. There was balling of fibers in the concrete supplied by the first truck. This was due to the wrong procedure adopted in adding and mixing the fibers in the truck. This procedure was corrected immediately and thereafter there was absolutely no balling of fibers.

All test equipment were taken to the construction site one day earlier and kept ready. The placing of fiber concrete pavement started at 8 AM in the morning. All the fresh concrete tests for the field concrete were done on the field by a team of students under supervision and ASTM Standards of testing were adopted wherever applicable. Beams of 6"x6"x21" (152.4x152.4x533.4mm) and 4"x4"x14" (101.6x101.6x355.6mm) and impact specimens 2.5"x6" (63.5x152.4mm) and cylinders 6"x21" (152.4x304.8mm) were cast for the determination of hardened concrete properties. The specimens were cast in steel molds using a mechanical vibrator for proper compaction of the concrete. After the casting was over, the specimens were kept covered with a plastic sheet until the next day when they were demolded and placed in lime saturated water for curing. The beams for fatigue test were painted with a curing compound (protex promulsion) after they were taken out of the curing tanks.

The north bound lanes of the pavement were placed on June 1, 1988 and the south bound lanes on the June 7, 1988. The concrete was supplied by a central ready mix plant, Rapid City, SD. A concrete paver GOMACO, IDA Groove, IOWA was used operating at a vibrator frequency of 8000 vibrations per minute. A rollerbug was used for finishing. After the rollerbug use, the surface was finished with long wooden floats. The finishability of the concrete was observed to be good because of the fly ash in the concrete. The pavement was then broom finished with a wide nylon brush. Because of lack of sufficient width of the broom, the sides were first hand finished and the central part was machine finished. Finally the finished surface was sprayed with the curing compound.

Weather conditions during construction:

The non-reinforced concrete pavement is 2419 ft. (737.31m) long and 7 1/2 in. (190.5mm) thick and includes a section cast with steel fiber reinforced concrete. The north bound lanes were cast on June 1 and the south bound lanes were cast on June 7, 1988. June 1 was almost an ideal day for concreting (62^{o} F and a relative humidity of 55%). June 7, though not intentionally selected, happened to be one of the most severe days for concreting with an ambient temperature reaching 105^{o} F with a relative humidity of 10%. These two conditions had provided us an opportunity to evaluate steel fiber reinforced concrete (SFRC) performance in two extreme weather conditions. There was about 55^{o} F variation between the hot day and the cool night temperature during the curing period. This condition created a considerable amount of thermal stress variation in addition to the normal drying shrinkage stress induced by the dry and hot weather conditions. This combined effect produced a considerable number (about 30) of shrinkage and thermal cracks in the non-reinforced concrete pavement section, particularly in the portion that was placed after 11 AM, when the temperature exceeded 100^{o} F. Only one crack appeared in the steel fiber reinforced concrete section.

Saw-Cut Joints :

The joints were cut as early as possible both in non-reinforced section and fiber reinforced section of the pavement. No problems were encountered in the non-reinforced section. However, when the joints were cut at approximately the same time interval, some raveling occurred and the fibers were pulled out. Later, when the joints were cut at the proper time, there was no raveling and the joints were alright.

CONSTRUCTION OF SHERIDAN LAKE ROAD PAVEMENT

Based on an extensive laboratory investigation, appropriate mixture proportions were selected to give the desired fresh and hardened concrete properties for the construction of SFRC pavement in the Sheridan Lake Road, Rapid City. Using the selected mixture proportions, the SFRC pavement on the south bound lanes were constructed on October 12, 1990. The construction started at 11 AM and was completed at 3 PM. The ambient temperature remained between 65 to 70°F throughout this period and the relative humidity was about 35 percent. The wind velocity was about 9-14 miles per hour. Thus the conditions were almost ideal for concrete construction.

The construction was completed without any problems. No balling of fibers occurred and the distribution of fibers in the concrete was satisfactory. Regular placement and finishing procedures were followed. A rollerbug was used to push the surface steel fibers down about 1/8 inch (3.175mm). Brush finish was used. The brush seemed to be new and it was stiffer than the previously used one. Therefore some mortar was dragged along with the brush.

Fresh concrete tests were made and the specimens were cast using concrete taken from the third truck and the last truck. Toward the end, during the construction of the last 20 ft. of pavement, the steel fibers were mixed in the field. The second batch samples were made with field-mixed concrete. The tests had indicated that the fiber content was slightly lower than specified.

The north bound lanes were paved on November 15, 1990. The same mixture proportions and the same procedures were used. The ambient temperature was a little cooler at 55°F and the relative humidity was the same (35% as on October 12). The wind velocity was 10 to 16 miles per hour. No problems were encountered in this construction.

MODIFICATIONS MADE FOR SFRC PAVING

The addition of steel fibers reduced the slump (apparent workability), and hence in order to maintain the same slump, the mixture proportions were modified. Fly ash was added to the mixture as an alternative to the addition of a superplasticizer noting the fact that acceptable workability could be obtained without having to resort to the recommended use of a superplasticizer. The workability of SFRC was not as good as the control mix but was still sufficient to allow the standard slipform (Haines Avenue) or formed (Sheridan Lake Road) paving operations to proceed with only minor adjustments.

The other modifications necessary for proper paving with SFRC are: (a) adding the fibers to the coarse aggregate and mixing to disperse the fibers prior to adding the sand, cement and water to prevent balling; (b) adjusting the vibration frequency of the spud vibrators to improve consolidation of the SFRC; (c) adjusting the tension on the astroturf drag to prevent pulling of the fibers onto the surface of the pavement; (d) requiring the broom finish to be applied in the same direction as the paving operation to minimize the exposure of steel fibers; (e) noting that tining is not recommended with steel fibers due to raveling.

In the case of SFRC, stricter quality control and additional care were needed during mixing and placing. Additional cure time was needed for SFRC as compared to plain concrete before joints could be saw-cut. Otherwise fibers tended to ravel joints when contacting the blades.

INSPECTION OF HAINES AVENUE PAVEMENT

Periodic field inspections were made to observe the crack formations, fiber protrusions, surface conditions, and any other distress in the pavement. Observations were made both on the steel fiber reinforced concrete and the non-reinforced concrete sections in order to have a comparative evaluation. On the right side the first inspection was made on June 6, and five subsequent inspections were done on June 16, 23, 30, July 7, and 25. During all the six inspections, not even a single crack was observed on the fiber reinforced concrete section and there were also no cracks on the non-reinforced concrete section of the pavement.

On the north bound lanes, the first inspection was made on June 16, a week after the completion of the pavement, and further field observations were made on June 23, 30, July 7, and 25.

During the first inspection it was found that only one crack had appeared in the fiber reinforced section while a number of cracks had appeared in the non-reinforced concrete section. The lengths of the cracks varied from 5 to 60 inches (127 to 1524mm) and the width of the cracks varied from 0.008 to 0.015 inches (0.20 to 0.40 mm). Mostly the cracks had appeared near the intersection of the longitudinal and transverse joints and they were in the middle of the pavement. Most of the cracks were thin hair line cracks and they formed very close and parallel to the transverse saw-cut joints.

During the second inspection, it was noticed that no further cracks had formed and the existing cracks did not widen or propagate further. All cracks were repaired and in two cases, where the cracks were longer and away from the saw-cut joints, segments (4x12 ft.) of the pavements were replaced with new concrete.

On September 30, 1988 the pavement was again inspected to observe any new crack formations. During the inspection no new cracks were found on the fiber concrete section, both on the north and south lanes of the pavement. There were also no new cracks on the north bound lanes of the non-reinforced section of the pavement. The north bound lanes were constructed on June 1, 1988 when the temperature was ideal for concreting.

On the north bound lanes, two new cracks were found, both parallel to the transverse saw-cut joints. These were thin hair line cracks, the first about 5 ft. 3 in. (1.6m) long and the second, 5 ft. 1 in. (1.55m) long. These cracks appeared to be shrinkage cracks and they were approximately 1 ft. to 1½ ft. (0.3 to 0.45m) away from the saw-cut joints. They both started from the middle longitudinal joint and extended towards the curb of the pavement. These cracks formed on that part of the pavement which was constructed on June 7, 1988 when the weather was most unfavorable. This particular portion of the slab was cast during the hottest time of the day.

Based on the comparison of the north and south bound lanes of the pavements, it is concluded that these cracks were the result of weather conditions, and not due to any inherent material defects. These cracks could have been avoided, or eliminated to a great extent by adopting appropriate precautionary measures, for hot weather concreting.

Fiber protrusions and surface fibers

The inspections had revealed that there were no fiber protrusions, or fiber sticking out at the surface. There were few (less than one fiber per square yard) fibers exposed at the surface ; they were lying flat. This is a very good achievement, much better than the anticipated result.

Further periodic inspections of the pavement at 3 month intervals were made until 30 April, 1993. During the inspection after one year (on June 27, 1989) no new cracks were found in the SFRC section. However in the unreinforced pavement there was a new crack, about 5.5 ft. (1.67m) long and about 0.15 inch (3.81mm) wide in the general location where most of the previous cracks had formed.

In the SFRC section there were no fiber protrusions or loose fibers on the pavement. However there were some fibers exposed, still attached to the pavement, lying flat. Near the surface where there was heavy wheel traffic, a careful inspection revealed that there were some steel fibers exposed and fully bonded to the matrix. They were bright and shiny indicating that they were undergoing the same wear as the coarse aggregate pieces. They easily merged with the concrete surface and it was not easy to distinguish them from the aggregate pieces. These fibers do not seem to cause any problems; they do not seem to be any nuisance or present a hazard. The normal wearing and spalling seen at the joints were the same for both the reinforced and unreinforced parts of the pavements.

No new cracks were noticed in the subsequent inspection till September 4, 1992 (51 months after construction). However some aggregate polishing (particularly at the junction of Haines Avenue and Van Buren street) took place due to the traffic. Along with the agregates, the bright shining steel fibers were also seen. However these fibers were bonded to the matrix just like the aggregates and they were wearing the same as the coarse aggregates. They were lying flat.

During the inspection on September 4, 1992, there were three new cracks. These were longitudinal (parallel to the traffic) and normal to the control joint and starting from the control joint. Two were on one side and one crack was on the other side of the control joint. These were wide cracks [one 3/16 inch (4.76mm) and two 1/8 inch (3.2mm)] and 2 to 5 ft. (0.6 to 1.52m) long. These cracks seem to have occured due to a distress in the sub-grade or a differential settlement at this location. These cracks were not due to any material inadequacy.

In the unreinforced section, some large cracks were also noticed.

On a subsequent inspection on March 4, 1993, (57 months after construction), a new crack of the same type and in the same location as described above was observed. However there was no widening or increase in the length of the old cracks during the past 6 months period. The pavement and these cracks were subjected to a prolonged and severe winter and a significant number of freeze thaw cycles. As stated before the location and nature of these cracks indicated that they were due to a settlement or some form of distress in the base material.

In general, it can be stated that after 60 months of service, the condition of SFRC pavement was satisfactory.

INSPECTION OF SHERIDAN LAKE ROAD PAVEMENT

The south bound lanes were inspected the next day after they were cast on October 13 at 1 PM. The transverse joints were already cut. There was no cracking in the pavement slab. There seemed to be a little raveling in the first and second joints and I was told that the transverse cutting was done about 16 hours after casting. Other joints were all right. The longitudinal cutting was done two days later. The slab was covered with poly sheets during the curing period because the night temperatures were below 30° F.

The slab was again inspected on October 20 and 24, 1990. There were no cracks. On October 24, the pavement was opened for traffic. The slab was again inspected on October 30 and November 10, 1990. There were no cracks and no other distress was noticed.

After the north bound lanes were completed, both sides were inspected on November 16, 18, 20, 25, and 30, 1990. Traffic was not yet allowed on the north bound lanes. There were no cracks and the pavement looked satisfactory. The north bound lanes were open for traffic in December 1990. This pavement was inspected periodically at 3 month intervals and the final inspection was made on April 30, 1993. This inspection took place after the pavement was subjected to a prolonged and severe winter and after a significant number of freeze-thaw cycles.

There were no cracks (not even a single crack was noticed in the FRC section), no spalling, and no popouts. There were no signs of any form of distress. There were a few fibers exposed on the surface; however these fibers were still attached to the matrix. They were not protruding; they were lying flat. In general, the pavement surface looked very good after 30 months of traffic.

TESTING FOR SKID RESISTANCE

Mr. Dan Johnston and Charles McFarling of the SD DOT conducted the skid resistance tests on the 29th of May, 1991. This was the first time skid resistance was measured for the Haines Avenue Pavement. This pavement was almost 3 years old. During the same run, the skid resistances were measured at two marked locations in the FRC and at three corresponding locations in the unreinforced portions of the pavement. In the FRC pavement, the locations included in both 5½ and 6 inch (139.7 and 152.4 mm) thick slabs. the measured skid numbers are given below. There is no recognizable difference in the skid resistance of the plain concrete and FRC pavements. After 3 years, the skid resistance was still satisfactory.

	SKID NUMBERS	
	South Bound Lane	**North Bound Lane**
FRC Pavement 1	39.4	44
FRC Pavement 2	37.0	39.2
Unreinforced 1	38.9	44.3
Unreinforced 2	43.2	42.8
Unreinforced 3	40.4	-

The skid resistance tests were also conducted the same day on the Sheridan Lake Road pavement. During the same run, the skid resistances were measured at 4 locations (two in the east bound and two in the west bound lanes) in the FRC pavements and similarly the skid numbers were obtained at four locations in the unreinforced (control) pavement. The measured skid numbers are given below:

	SKID NUMBERS	
	East Bound Lane	**West Bound Lane**
FRC Pavement 1	39.0	43.6
FRC Pavement 2	43.8	50.7
Unreinforced 1	40.2	41.0
Unreinforced 2	44.5	43.0

There is no significant difference in the skid numbers for the FRC and unreinforced concrete pavements.

CORE TEST RESULTS

The Haines Avenue steel fiber reinforced section has two thicknesses, 5 inch (127 mm) and 6 inch (152.4 mm), whereas the plain concrete section has 7.5 inch (190.5 mm) thickness. Cores were taken to verify the thickness of the steel fiber reinforced concrete pavement, to check for the consolidation and fiber distribution, and to determine the compressive strengths.

A total of twelve cores were taken from both the sides of the pavement, six from 6 inch (152.4 mm) and six from 5 inch (127 mm) pavement sections ; six were from the north lanes and other six were from the south lanes. The measured dimensions were never less than 5 and 6 inch (127 and 152.4 mm) thick at specified locations. The observed variations in the depth are within permissible limits.

An inspection of the cores had shown that the consolidation of the concrete was good. There were no large honeycombs or voids. The fiber distribution across the depth seemed satisfactory.

These cores were tested according to ASTM C142, and all the details are given in the Table 3 & 4. The compressive strengths were calculated using the ASTM recommended reduction factor. The calculated values indicate that the minimum strength is 4400 psi (30.34 MPa) and the maximum strength is 5660 psi (39.03 MPa). The mean value is 5440 psi (37.51 MPa) with a standard deviation of 420 psi (2.89 MPa) and coefficient of variation of 8 percent. The results indicate that the steel fiber concrete in place was satisfactory.

In the Sheridan Lake Road pavement, cores were also taken from both sides of the pavements. Four cores were taken from the south lanes of SFRC pavement at distances of 183 ft. (55.78 meters) and 219.8 ft. (67 meters) from the starting point of the SFRC pavement which was placed on October 12, 1990. Four more cores were taken from the north bound lanes of SFRC pavement at distances of 212.9 ft. (64.9 meters) and 288.7 ft. (88 meters) from the starting point. The measured dimensions and test results are given in Supplementary Report 2. The actual thicknesses were satisfactory and they were not less than 6 inch (152.4mm). The consolidation of the concrete was good and there were no large voids or honeycombs. The fiber distribution across the depth was satisfactory. The lowest compressive strength obtained was 4130 psi (28.48 MPa) and the highest compressive strength obtained was 6090 psi (42.00 MPa). The mean value was 5190 psi (35.79 MPa) with a standard deviation of 800 psi (5.52 MPa) and coefficient of variation of 15 percent.

COST COMPARISON

The bid prices quoted by the successful contractors for the Haines Avenue and Sheridan Lake Road pavements construction are given below:

Haines Avenue pavement:

Item	Unit	Cost per unit
Portland cement concrete (PCC) 7-5" thick non reinforced	sq.yd.	$19.25
PCC 5" thick steel fiber reinforced concrete (Hooked-end fibers)	sq.yd.	$25.00
PCC 6" thick steel fiber reinforced concrete (Hooked-end fibers)	sq.yd.	$28.00

Sheridan Lake Road pavement:

PCC 8" thick non-reinforced	sq.yd.	$20.00
PCC 5" thick steel fiber (corrugated) reinforced concrete	sq.yd.	$25.00
PCC 6" thick steel fiber (corrugated) reinforced concrete	sq.yd.	$28.00

Both contractors have not constructed steel fiber reinforced concrete pavements before and in both cases, they were treated as experimental constructions. The SFRC in both cases were only small quantities about 5% of the total pavement concrete. It is interesting to note that the market price of the corrugated steel fiber at that time was only about 60 percent of the hooked end steel fiber. However, the prices quoted for SFRC by the contractors were the same for using both types of fibers. It is recognized that it is misleading to compare costs associated with experimental constructions.

DISSEMINATION OF INFORMATION

Accepting an invitation from the Chairman of the Transportation Research Board Committee on Mechanical Properties of Concrete A2E03, a brief presentation was made by V. Ramakrishnan and Dan Johnston, South Dakota Department of Transportation, about the details of design and construction of the Haines Avenue steel fiber reinforced concrete (SFRC) pavement. The presentation titled "The Construction of Steel Fiber Reinforced Concrete Highway Pavement" was made on January 24, 1989 at the TRB Committee meeting, Sheraton Washington Hotel, Washington D.C. Slides depicting the actual construction procedures (including the finishing process), the performance of SFRC during placement, and the measured properties of the hardened concrete were shown to the committee members and visitors (about 50 concrete engineers). The presentation was well received and an interesting discussion followed. No apprehension was expressed and no adverse comments were made. They are interested to know about the future performance of the pavement.

A research paper entitled "Field Performance of Fiber Reinforced Concrete Highway Pavements" authored by V. Ramakrishnan, Larry Kostaneski and Dan Johnston was presented at the First Materials Engineering Congress, organized by the American Society of Civil Engineers, in Denver, CO, 1990, and also published in the proceedings, Vol. 2, pp. 903-912.

CONCLUSIONS

1. The steel fiber reinforced concrete (with 0.5 volume percent of fibers) pavements could be built with the conventional equipment without any need to invest in any additional equipment. However additional care is needed during mixing and placing.

2. The core test results showed that the hooked end steel fiber reinforced concrete in place had much higher compressive strength than the specified strength, the fiber distribution was uniform, and fiber reinforced concrete was well compacted without voids.

3. The inspection revealed that the number of cracks observed were much more numerous in the non-reinforced section of the pavement. Some of them were due to the hot weather conditions which could have been eliminated if necessary precautionary measures were taken. Some of the cracks were due to shrinkage of the concrete. SFRC provides better performance, specially if less than perfect conditions exist in the field.

4. There were no fiber protrusions or fiber sticking out at the surface. There were very few (less than one fiber per square yard) fibers exposed and they were lying flat.

5. The periodic inspection of the pavement indicated that the behavioral performance of hooked end steel fiber reinforced concrete, with 0.5 volume percent of fibers, is far superior to that of plain non-reinforced concrete used in the construction of Haines Avenue pavement. Corrugated steel fiber reinforced concrete with 0.5 volume percent of fibers used in the construction of Sheridan Lake Road pavement also performed satisfactorily.

6. The SFRC pavement sections with considerably reduced thickness (20 percent and 33.3 percent) performed equally well compared to the non-reinforced concrete [7.5 inch (190.5mm) thick] pavements.

7. The skid resistance numbers were almost the same for both plain concrete and SFRC pavements.

8. Additional cure time was needed for SFRC as compared to plain concrete before joints can be saw-cut. Otherwise fibers tend to ravel joints when contacting the blades.

9. Based on the enhanced properties of the SFRC compared to plain concrete, it can be stated that the future maintainance cost of SFRC pavement will be less resulting in an overall long-term economy.

10. After the experience gained in the construction of SFRC pavements, both contractors and their construction personnel including the concrete finishers felt comfortable and confident in working with SFRC. Their initial apprehension and resistance were dispelled.

RECOMMENDATIONS

1. Based on the success of this experimental project, it is recommended that fiber reinforced concrete (FRC) could be used for the construction of pavements. This will enhance the performance and structural efficiency of the pavement with a potential for

longer life. FRC pavements could be used in all highways, urban, or rural or interstate highways, either with high density or low-density traffic.

2. When FRC is used for the construction of pavements, the thickness can be reduced without compromising its strength or performance. It is recommended that a 20 percent reduction in thickness can be made to reduce the total cost of the pavement. This is a conservative recommendation considering the fact that in the experimental project, the SFRC pavement with 33.3 percent reduced thickness performed well. However this reduction should depend on the fiber content, the type and efficiency of the fibers used. It is possible to achieve further reduction with higher fiber content or with more efficient fibers.

3. When steel fibers are used, it is recommended that a minimum of 0.5 volume percent of fibers should be used in the case of efficient fibers. Higher quantities of fibers should be used in the case of less efficient fibers. It is recommended that the basic properties and the performance characteristics, particularly the performance when subjected to fatigue loading, should be evaluated for the proposed or selected mixture proportion for the FRC.

4. It is recommended that strict quality control measures should be followed when FRC is used in the pavement construction.

5. Additional research is needed to optimize the fiber content in the concrete in regard to the selected type and aspect ratio of the fiber to achieve additional reduction in thickness and/or to enhance the static and fatigue flexural strengths.

ACKNOWLEDGEMENTS

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Without their valuable assistance this work would not have been successfully completed. The author wishes to thank SDDOT for sponsoring this project. Numerous graduate students have contributed considerable amount of time in conducting the experiments. The author wishes to thank all of them and particularly three graduate students, Sidhesh Kakodkar, P. S. Punit and Phani Kumar Mukhara.

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TABLE 1 MIX DESIGNATION

MIX #	TYPE OF MIX	FIBER TYPE	VOLUME % OF FIBER
SD - I	Trial Mix	Hooked end steel (ZC-60/0.80)	0.5 (66 lbs/cu.yd)
SD - II	Trial Mix	Hooked end steel	0.5
SDP - I	Actual Mix	Hooked end steel	0.5
SDP - II	Actual Mix	Hooked end steel	0.5
SDP - IP	Actual Mix (Plain)	-	-

TABLE 2 BASIC MIX PROPORTIONS FOR THE FIELD MIX SDP

Fine Aggregate	i) For mix SDP-I: 1331 lbs/cu. yd. ii) For mix SDP-II:1392 lbs/cu. yd.
Coarse Aggregate ***	i) For mix SDP-I: 1634 lbs/cu. yd. ii) For mix SDP-II:1574 lbs/cu. yd.
Cement (Type I/II)	i) For mix SDP-I: 525 lbs/cu. yd ii) For mix SDP-II:527 lbs/cu. yd
Fly-ash	i) For mix SDP-I: 113 lbs/cu. yd ii) For mix SDP-II:113 lbs/cu. yd
* Water	i) For mix SDP-I: 252.00 lbs/cu. yd. ii) For mix SDP-II:259.65 lbs/cu. yd.
Air-entraining agent:	i) For mix SDP-I: 352.94 cc(12 oz)/yd3 ii) For mix SDP-II:392.94 cc(13.4oz)/yd3

** Fiber: Hooked end steel fibers 66 lbs/cu. yd.

Notes:

* The water cement ratio used for mix SDP-I was 0.48. The water to cement plus fly ash ratio was 0.39. For mix SDP-II, additional water of 0.82 gallons and air entraining agent of 40 cc (neutralized vensol resin) were added in order to increase the workability and the air content. The water cement ratio for the mix SDP-II was 0.49. The water to cement plus fly ash ratio was 0.41.

** Hooked end steel fibers ZC 60/0.80 (60 mm long and 0.80 mm diameter) were used.

*** A mixture of 1 1/2 in. and 3/4 in. coarse aggregates was used.

TABLE 2A COMPARISON OF PROPERTIES FOR CONTROL CONCRETE AND SFRC

Concrete	Unit Weight Lb / ft^3	Static Modulus x 10^6 psi	Comp. Strength psi	Pulse Velocity ft / sec.	First Crack Strength psi	Modulus of Rupture psi
Control	148	5.6	5740	15255	752	752
SFRC	151	5.4	5800	15350	882	910

Concrete	Fatigue Strength psi	Endurance Limit psi	Impact Strength	
			First Crack	Ultimate Failure
Control	415	55 %	15	17
SFRC	655	72 %	22	118

Concrete	ASTM Toughness Indices			Ratio	
	I5	I10	I30	I10 / I5	I30 / I10
Control	1	1	1	1	1
SFRC	4.59	8.91	23.23	1.93	2.604

Table 3 Test Results of Haines Avenue pavement Cores

Tested as Per ASTM C42-84a.

Specimen No.	Dimensions (in inch)				Remarks
	Before Trimming		After Trimming		
	Length	Dia	Length	Dia	
1) 34 + 75 14' RT C/L M1669(22) PENN	6.0	4.02	5.9	4.02	No voids, no honeycombing were found in the specimen. Compaction and fiber distribution are found to be good.
2) 35 + 32 6' RT C/L M1669(22) PENN	6.40	4.02	6.0	4.02	No voids, no honeycombing were found in the specimen. Compaction and fiber distribution are found to be good.
3) 35 + 50 20' RT C/L M1669(22) PENN	6.6	4.02	5.8	4.02	No voids, no honeycombing were found in the specimen. Compaction and fiber distribution are found to be good.
4) 35 + 75 4' RT C/L M1669(22) PENN	5.5	4.02	5.0	4.02	No voids, no honeycombing were found in the specimen. Compaction and fiber distribution are found to be good.
5) 35 + 97 10' RTC/L M1669(22) PENN	5.7	4.02	5.2	4.02	No voids, no honeycombing were found in the specimen. Compaction and fiber distribution are found to be good.
6) 36 + 40 18' RTC/L M1669(22) PENN	5.3	4.02	5.1	4.02	No voids, no honeycombing were found in the specimen. Compaction and fiber distribution are found to be good.

Table 3A Test Results of Haines Avenue pavement Cores

Tested as Per ASTM C42-84a.

Specimen No.	Dimensions (in inch)				Remarks
	Before Trimming		After Trimming		
	Length	Dia	Length	Dia	
1) 34 + 70 20' LT C/L M1669(22) PENN	6.1	4.02	5.9	4.02	No voids, no honeycombing were found in the specimen. Compaction and fiber distribution are found to be good.
2) 35 + 15 14' LT C/L M1669(22) PENN	6.1	4.02	5.8	4.02	No voids, no honeycombing were found in the specimen. Compaction and fiber distribution are found to be good.
3) 35 + 43 6' LT C/L M1669(22) PENN	6.3	4.01	5.9	4.01	No voids, no honeycombing were found in the specimen. Compaction and fiber distribution are found to be good.
4) 35 + 72 9' LT C/L M1669(22) PENN	5.1	4.02	5.1	4.02	No voids, no honeycombing were found in the specimen. Compaction and fiber distribution are found to be good.
5) 35 + 88 17' LT C/L M1669(22) PENN	5.3	4.02	5.1	4.02	No voids, no honeycombing were found in the specimen. Compaction and fiber distribution are found to be good.
6) 35 + 98 20' LT C/L M1669(22) PENN	5.1	4.01	5.0	4.01	No voids, no honeycombing were found in the specimen. Compaction and fiber distribution are found to be good.

NOTE: Photographs of the specimens were taken before trimming and after testing. Copies of those photographs are attached.

Table 4 Compressive Strength of Haines Avenue pavement Cores

Tested as Per ASTM C42-84a.
Date of Test: 07/14/88

Specimen No.	Age (Days)	Dimensions Length (inch)	Dimensions Dia. (inch)	Length / Dia.	Maximum load (lbs)	Strength correction factor	Compressive Strength (psi)
1) 34 + 75 14' RT C/L M1669(22) PENN	44	6.2	4.02	1.54	74,500	0.96	5,640
2) 35 + 32 6' RT C/L M1669(22) PENN	44	6.3	4.02	1.57	67,000	0.97	5,120
3) 35 + 50 20' RT C/L M1669(22) PENN	44	6.1	4.02	1.52	72,000	0.97	5,500
4) 35 + 75 4' RT C/L M1669(2) PENN	44	5.2	4.02	1.29	61,000	0.97	4,660
5) 35 + 97 10' RT C/L M1669(22) PENN	44	5.4	4.02	1.34	57,500	0.97	4,400
6) 36 + 40 18' RT C/L M1669(22) PENN	44	5.5	4.02	1.37	67,500	0.97	5,160

Table 4A Compressive Strength of Haines Avenue pavement Cores

Tested as Per ASTM C42-84a.
Date of Test: 07/14/88

Specimen No.	Age (Days)	Dimensions Length (inch)	Dimensions Dia. (inch)	Length / Dia.	Maximum load (lbs)	Strength correction factor	Compressive Strength (psi)
1) 34 + 70 20' LT C/L M1669(22) PENN	38	6.2	4.02	1.54	67,000	0.96	5,070
2) 35 + 15 14' LT C/L M1669(22) PENN	38	6.2	4.02	1.44	75,500	0.95	5,650
3) 35 + 43 6' LT C/L M1669(22) PENN	38	6.2	4.02	1.54	65,500	0.96	4,960
4) 35 + 72 9' LT C/L M1669(2) PENN	38	5.3	4.02	1.32	70,500	0.94	5,220
5) 35 + 88 17' LT C/L M1669(22) PENN	38	5.4	4.01	1.35	63,000	0.94	4,690
6) 35 + 93 20' LT C/L M1669(22) PENN	38	5.3	4.01	1.32	76,000	0.94	5,660

Mean 5144 psi

Standard deviation 419 psi

Coefficient ofvariation 0.08

STEEL FIBER REINFORCED PORTLAND CEMENT CONCRETE PAVEMENT
SFRC

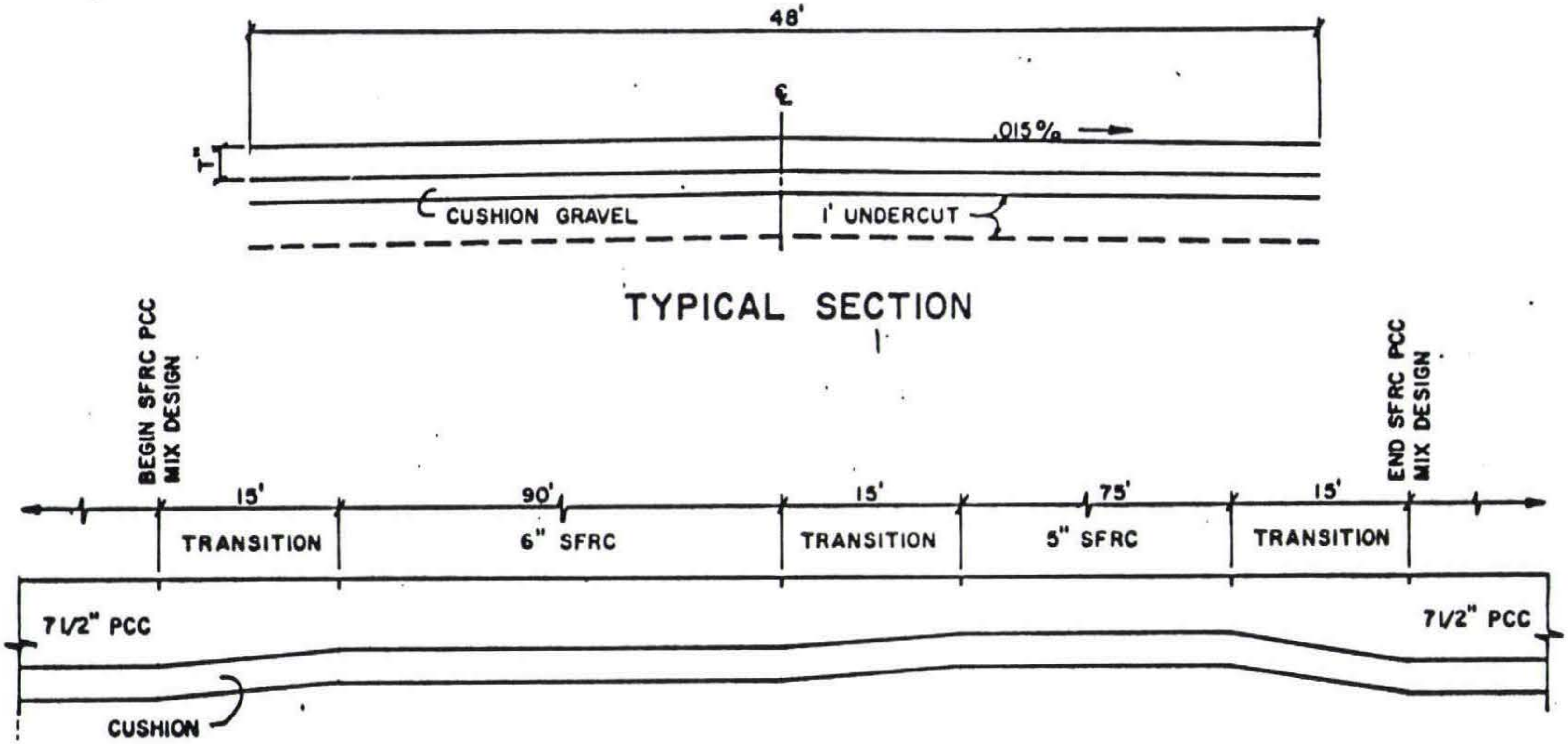

FIG.1 PAVING PROFILE

NOTES:

CONTRACTOR SHALL PAVE CONTINUOUSLY USING TRANSITIONS AS SHOWN.

CUSHION GRAVEL & UNDERCUT THICKNESSES SHALL BE AS SHOWN FOR T" PCC PAVEMENT

TRANSVERSE JOINTS SHALL BE AT 15' CENTERS, LONGITUDINAL JOINTS AT 12' CENTERS UNLESS OTHERWISE DIRECTED BY THE ENGINEER.

STATIONS OF TRANSITIONS AND SFRC SECTIONS SHALL BE STAKED BY THE ENGINEER.

SD87-02-F

SD Department of Transportation
Office of Research

Quality Control and Evaluation of Fiber Reinforced Concrete

Study SD87-02
Final Report

SUPPLEMENTARY REPORT-2
CONSTRUCTION OF SHERIDAN LAKE ROAD PAVEMENT

Prepared by
South Dakota School of Mines & Technology
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Rapid City, SD 57701-3995

May, 1993

SD87-02-F

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ABSTRACT

This report presents the experimental and analytical investigation of performance characteristics of corrugated steel fiber reinforced concrete used for the construction of the pavement at Sheridan lake road in Rapid city, South Dakota.

All the experiments were conducted using the appropriate ASTM standards. The test program for fresh concrete properties consisted of the following tests: slump, inverted cone time, air content, unit weight, and concrete temperature. The following hardened concrete properties were also determined: compressive strength, elastic modulus, static flexural strength (modulus of rupture), load-deflection curves, first crack toughness, post-crack behavior, ASTM toughness indices, flexural fatigue strength and endurance limits.

The test results indicate that there was no balling or tangling of fibers during mixing and placing. There was an appreciable increase in post crack energy absorption capacity and ductility due to addition of fibers. When compared to plain concrete there was a significant increase in the fatigue strength and endurance limit. The results of statistical analysis indicate that concrete reaches endurance limit at two million cycles.

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1. INTRODUCTION

Concrete is the most widely used construction material in the world. It has many advantages such as relatively low cost, availability, easy adaptability, and versatility of use. It is also one of the most variable with regards to material properties (3*). Its low tensile strength capacity and brittle failure tendency are the two major problems to overcome in the design. These deficiencies are commonly circumvented by providing conventional steel reinforcement or applying prestress in concrete member.

The concept of reinforcing brittle building materials has been practiced from the early civilizations. In the past two decades there have been revolutionary changes in the usage of concrete. The overall behavior of concrete is improved by adding fibers. Some of the commonly used fibers are asbestos, glass, polypropylene, plastic, and steel. The American Concrete Institute defines fiber reinforced concrete as "Concrete made of hydraulic cements and containing fine or fine and coarse aggregates along with discontinuous discrete fibers" (13). The primary function of fibers is to arrest any advancing cracks, thus delaying their propagation across the concrete matrix. The ultimate strain capacity of the composite is thus increased compared to that of the unreinforced matrix. Thus, due to the addition of the fibers, concrete performs in a more ductile manner (2,6,7,9,10,11).

* - Referred in chapter eight.

The addition of fibers in the concrete matrix has many important effects. Most notable among the improved mechanical characteristics of fiber reinforced concrete (FRC) are its superior fracture resistance and resistance to impact and impulsive or dynamic loads compared to plain concrete (2,4,7). In fiber reinforced concrete, thousands of small fibers are randomly dispersed into the concrete and thus improve concrete properties in all directions. Research has shown that addition of steel fibers in concrete improves tensile strength, interfacial bonds, and impact strength (2,4,9,11,12). However, the increase in these properties will vary from substantial to nil depending on the quantity and the type of fibers used.

Use of steel fiber reinforced concrete offers improved pavement life for a given slab thickness or significantly reduced thickness for overlay or pavement for equivalent performance (9,16).

The superior properties of fiber reinforced concrete have led to a gradual increase in its applications in many structural areas (2,3,9,10,11). The large increase in fatigue strength finds application in pavements, machine foundations, marine structures, overlays at airport runways and highways subjected to cyclic loading. Increased impact resistance, toughness, static flexure, and ductility may find applications in tunnel lining, nuclear reactor shielding, pile caps, and earth-quake resistante structures.

2. LITERATURE SURVEY OF THE BEHAVIOR OF PLAIN AND FIBER REINFORCED CONCRETE SUBJECTED TO FATIGUE LOADING

Fatigue is a process of progressive, permanent internal structural damage in a material subjected to repetitive stresses, and is a principal mode of failure in structural and mechanical systems (32,36). Concrete bridges, offshore structures, pressure vessels, cranes, and concrete pavements are some of the structures most influenced by repetitive fatigue loading.

2.1 S-N Curve

The classical approach to fatigue has focused on the S-N diagram which relates fatigue life (Number of cycles to failure, N) along abscissa to cyclic stress amplitude, S_a(or cyclic stress range S_r) along ordinate, Fig.1 (37). This curve reveals a linear relationship between flexural fatigue strength and number of cycles; after reaching endurance limit the line becomes parallel to the X-axis (16,24,29). S-N curves enable prediction of mean fatigue life of concrete under constant-amplitude cyclic stress. This constant amplitude S-N diagram can be used to relate stress to either crack initiation period or total fatigue life. Fatigue tests usually exhibits a large scatter, and it is necessary to test a number of specimens at each several stress levels in order to establish the S-N curve of a particular concrete.

2.2 Cumulative damage theory

Palmgren (36) first devised a cumulative damage concept to explain the fatigue behavior of engineering materials in

1924. Miner (36) extended this concept to propose a linear damage theory. The well known Miner's linear damage theory is still being widely used due to its simplicity.

In accordance with Siemes (36) "The most commonly employed hypothesis for determining the degree of fatigue damage due to random stresses is Miner's hypothesis. This rule establishes a relation between the amount of damage and the results of constant-amplitude tests. The conception is that the contribution to the fatigue damage in consequence of a single stress cycle ranging from a minimum stress $S'_{min\ i}$ to a maximum stress $S'_{max\ i}$ has a magnitude of $1/N_i$, where N_i denotes the number of cycles which results in fatigue failure, in a constant amplitude test at the same level of stress, provided that the conditions pertaining to the single cycle have to be the same as those pertaining to the constant amplitude test", as shown in Fig.2 (36).

According to Miner's rule the damage contribution M due to a number of C stress cycles is the sum of the damage contributions of each of individual cycle (29,36). The rule is called a cumulative damage rule.

$$M = \sum_{i=1}^{c} 1/N_i \qquad \text{.......................} \quad (1)$$

The rule is moreover linked to a failure criterion, as it is presupposed that failure occurs when the total damage M becomes equal to unity.

$$M = \sum_{i=1}^{c} 1/N_i = 1 \qquad \text{.....................} \quad (2)$$

2.3 Fatigue load spectrum

Fatigue failure occurs when a concrete structure fails catastrophically at less than design load after being exposed to a large number of stress cycles.

The spectrum of flexural fatigue load cycles is shown in Table 1 (20). This classification gives the approximate boundaries for various applications of fatigue loading. It also defines the approximate range for the terms "low-cycle fatigue," "high-cycle fatigue," and "superhigh-cycle fatigue" (20).

2.4 Endurance limit

In many applications, particularly in pavements, bridge deck overlays, and offshore structures, design procedures currently in use consider only the modulus of rupture of concrete in determining it's fatigue life. Plain concrete has a fatigue endurance limit (strength at 2,000,000 cycles) of 50 to 55% of its static flexural strength (16,20,22). Endurance limit in flexure decreases with increase in the number of load cycles. Beyond 10 million cycles there is no reliable data for plain concrete. Most of the investigations indicate that plain concrete does not exhibit an endurance limit for an infinite number of cycles (16). A properly designed fiber reinforced concrete (FRC) can achieve 90 to 95 percent endurance limit. The endurance limits for fiber reinforced concrete has been defined by Ramakrishnan (16) as:

1. Endurance Limit : Percentage of modulus of rupture of plain concrete.

The endurance limit is defined as the maximum fatigue flexural stress at which the beam could withstand two million cycles of non-reversed fatigue loading, expressed as a percentage of modulus of rupture of plain concrete (16).

2. Endurance Limit: Expressed as a percentage of its modulus of rupture.

The endurance limit of concrete can also be defined as the fatigue stress at which the beam could withstand two million cycles of non-reversed fatigue loading, expressed as percentage of its modulus of rupture (16).

Tatro(20) refers to the endurance load as the maximum load at which an infinite number of cycles can be sustained, which is normally termed "fatigue limit" in metal hand book.

Theoretically, with a higher endurance limit, the concrete cross section of the pavement could be reduced. Alternatively, using the same cross section of the pavement could result in a longer life span or higher load carrying capacity or both.

2.5 Variables that affect the performance of concrete subjected to fatigue loading (17)

Fatigue behavior of FRC is known to depend on a number of intrinsic and extrinsic factors, of which the former comprise properties of component materials while the later concern the dimensions of the test specimen and the method of testing. Table 2 shows the various parameters affecting the performance of fiber reinforced concrete subjected to flexural fatigue loading.

2.5.1 Air content

Air-entrained concrete is recommended for all structures under conditions of severe exposure and for all pavements regardless of climatic conditions. Entrained air benefits concrete mainly in two different ways: (1) improved resistance of the concrete to freezing and thawing and (2) improvement of the workability and decrease in segregation of freshly mixed concrete. If there is no change in water cement ratio, the flexural fatigue strength of concrete will be reduced by 3 to 5% for each percent air added (24,30).

Air-entrained concrete exhibits longer fatigue life at low stress ratios (less than 77% of static compressive strength) and shorter fatigue life at higher stress levels. Flexural fatigue strength decreases as air content increases; this is illustrated in Fig.3 (30). A higher percent of air weakens the bond between the cement paste matrix and aggregate. As the air content increases, the failure of concrete occurs increasingly at the aggregate cement paste boundary (30).

2.5.2 Water-cement ratio

The fatigue behavior of plain concrete in flexure is affected by the water-cement ratio of the concrete. Dan-Ynn Lee and F.Wayne (30) found that the fatigue strength decreases for low water-cement ratio (0.32) concrete, but no discernible difference in fatigue strength for concretes with a water cement ratio in the range 0.4 to 0.6 was found.

2.5.3 Aggregate

The fatigue behavior of concrete in flexure is affected by the coarse aggregate type used in the concrete. At high stress levels (greater than 77% static compressive strength) the concrete made with natural gravel (rounded) exhibits a higher fatigue strength than that exhibited by the concrete made with crushed limestone (sharp edged). At low stress levels there does not seem to be a significant difference in fatigue strength (17,30). In normal weight concrete (NWC), the aggregate is substantially stiffer than the mortar, and cracking occur first at the interface between mortar and aggregate and propagates into the mortar. Aggregate with rough surface texture provides a better bond between the aggregate particles and the paste. Flexural fatigue strength of concrete is not significantly influenced by the type of fine aggregate used (17,24,30).

2.5.4 Pozzolanic admixtures

The addition of silica fume to light weight aggregate increased its flexural fatigue life by 60 to 80 percent, compared to light weight aggregate (LWA) and normal weight aggregate (NWA) concrete of similar strength. The same effect was not found in NWA concrete. The addition of silica fume to LWA concrete improved its flexual bond by 100 percent (17).

2.5.5 Rate of Loading

Concretes tested at accelerated rates under laboratory conditions may have higher flexural fatigue strength than similar concretes subjected to much slower service loading. At high levels of loading, accelerated fatigue tests can significantly over estimate the flexural fatigue strength of concrete which may be loaded at much slower rate in service.

The effect of loading has been studied by many investigators. A common conclusion is that frequency of loading between 50 and 900 cycles per minute has little effect on flexural fatigue strength, provided that the maximum stress level is less than about 75 percent of the static strength (17,24,35). For higher stress levels, a significant reduction on flexural fatigue strength with decreasing rate of loading has been observed, see Fig.4.

2.5.6 Rest period

The effect of rest period during the cyclic process was studied in connection with flexure fatigue by Hilsdorf and Kesker(17), who reported that when allowing 5 minutes rest periods every 4500 cycles, the concrete seemed to recover and raise the fatigue life. Longer rest periods did not seem to improve the fatigue life further (17).

2.5.7 Range of stresses

The effect of range of stress may be illustrated by the stress-fatigue life curves, commonly referred as S-N curves, shown in Fig. 5, the ordinate is the ratio of the maximum stress, S_{max}, to the static strength. In this case, S_{max} is computed flexural tensile stress, and the static strength is the modulus of rupture stress, f_r. The abscissa is the number of cycles to failure, plotted on a logarithmic scale.

Curves a and c indicate that the fatigue strength of concrete decreases with increasing number of cycles. The influence of load range can be seen from comparison of curves a and c in Fig.4. These curves were obtained from tests with load ranging between a maximum and a minimum which was equal to 75 or 15 percent of the maximum, respectively. It is evident that decrease of the range between maximum and minimum results in increased flexural fatigue strength for a given number of cycles (20,24).

2.5.8 Stress gradient

Stress gradient is the variation of flexural stress along the depth of FRC section, when the extreme fiber stress is maximum. Stress gradient influences the flexural fatigue strength of concrete; in accordance with the results of static tests the strain gradient retards internal microcrack growth. When specimen is eccentrically loaded, there are regions stressed to levels lower than extreme fiber stress. Therefore, the specimen can be subjected to a greater number of cycles than a uniform stressed specimen at the same maximum stress level. Flexural fatigue strength of eccentric specimen is 15 to 18 percent higher than for uniform stressed specimen for a flexural fatigue life of 40,000 to 1,000,000 cycles, this is illustrated in Fig.6 (24).

2.6 Available models

In 1970, Aas-Jakobsen (22) proposed an equation where the role of the stress ratio R on the fatigue strength was recognized. He showed that the relationship between f_c^{max}/f'_c and f_c^{min}/f'_c is linear for fatigue failures at $N = 2X10^6$ load repetitions. This implies that the relationship between f_{max}/f'_c and R (f_{min}/f_{max}) should also be linear. Aas-Jakobsen combined this relationship between f_{max}/f'_c and R with the linear relationship between f_c^{max}/f'_c and log N, and he derived general f-N-R relationship (22,33).

$$f_c^{max}/f_c' = 1-\beta(1-R)*\log_{10}N \;.............(3)$$

where $R = f_{min}/f_{max}$ and β is the material coefficient.

Aas-Jakobsen gave the value β=0.064. This equation can be used for $0 \le R \le 1$, but not for stresses which alternate between compression and tension. Tepfers-Kutti (23) has shown this equation is valid for fatigue of concrete when subjected to compression stresses, tensile stresses and flexural stresses.

Rush (20) had demonstrated convincingly that the sustained strength of concrete is time dependent, and the long-time strength may approach 75 percent of the short-time static strength of concrete tested under ASTM loading rate. The Aas-Jackobsen equation doesn't include the rate of loading a variable. The omission of variable is unacceptable in low-cycle fatigue range.

Thomas T.C. Hsu (20) extended Aas-Jakobsen equation by introducing the element of time, proposed new equations for flexural fatigue strength that would express the four variable relationship (f-N-T-R), Fig.7 (20).

For high-cycle fatigue:

fmax/fc' = 1-0.0662 (1-0.556 R)*log N -0.0294 log T..(4)

For low-cycle fatigue:

fmax/fc'= 1.20 - 0.20R - 0.133 (1-0.779 R) log N -

0.053 (1-0.445 R) log T .... (5)

Boundary between low-cycle and high-cycle fatigue:

fmax/fc' = 1 + (0.0229+0.0490 R) log T .... (6)

The Classical model (The Basquin equation) (38)

$$NS^m = C \quad \text{.......................} \quad (7)$$

in which S = stress amplitude s_a, or stress range S_R; and m and C are empirical constants. This model is valid for high cycle range.

For low cycle fatigue (less than 10^4 cycles) the following model is valid

$$\delta_e/2 = \sigma'_f/E*(2N)^b + \epsilon'_f(2N)^c \quad \text{..............} \quad (8)$$

in which σ_e = the strain range, E = modulus of elasticity and other terms are empirical constants.

Langer model

A strain range cycle to failure model proposed by Langer was used to develop design curves in the American Society of Mechanical Engineers and Pressure Vessel Code. The S-N curve is of the form (38):

$$S = B*N^{-1/2}+S_e \quad \text{.....................} \quad (9)$$

in which B and S_e empirical constants

$$S = \tfrac{1}{2}(\delta\epsilon)*E$$

In the linear elastic range, S is the actual stress amplitude. In general, the empirical constants in the fatigue laws depends on the material and are significantly affected by environments (corrosion, cyclic frequency, etc.)

After several decades of experimental research and preparation of comprehensive review of current knowledge, it is seen that there is little fatigue data available to the designer, Butler et al. (39) conducted a test on over 50 beams 220x200x1500 mm (8.8x8x6 in) reinforced with 1.2, 2.3, and 3.3% volume of steel fiber. The steel fibers used in their investigation were melt-extracted stainless steel fibers (25X0.5mm), but also some supplementary testing of concretes containing hook ended (40x0.4mm) and indented fibers (50X0.5mm) were used. The test specimens were loaded with triangular wave from varying between a minimum stress level (S_{min}) of 10 percent of S_{max} to a selected maximum stress level S_{max}, at frequencies ranging from 4Hz to 20Hz.

On the basis of their investigation, Butler et al., developed a graph for the prediction of the fatigue life of a test specimen this is illustrated in Fig.8. In this graph, the stress is obtained by deriving a ratio of the maximum stress applied and the ultimate flexural strength.

Since the fatigue strength of fiber reinforced concrete is highly dependent on the fiber volume, Ramakrishnan (16) conducted a comparative evaluation of fatigue properties of four different fiber types (hooked end steel, straight steel, corrugated steel, and polypropylene) in two different quantities (0.5 and 1.0% by volume) using the same basic mix proportions for all concretes. Third point loading was used in the test for the determination of flexural and fatigue strength. The test beams were of dimensions 152 X 152 X 457 mm (6x6x18 inch) and were subjected to non-reversed fluctuating loads at a frequency of 20 Hz (16).

On the basis of Ramakrishnan's investigation, two graphs were developed for the prediction of the fatigue life of a test specimen, this is illustrated in Fig.9 and Fig.10. In the graph, the stress level f_{fmax}/f_r' obtained from the graph times the modulus of rupture of the investigating mix. Fiber types are as follows:

Type A: Two-inch long hooked end steel fibers with an aspect ratio of 100.

Type B: Low carbon straight steel fibers with an aspect ratio of 100.

Type C: Two-inch long corrugated steel fibers with an application, although there still may be some initiation period during which the material at the tip of the flaw undergoes dislocation pile up, miscarried formation, etc., prior to the onset of cycle-by-cycle growth (38).

The basic parameter of fracture mechanics analysis is the stress intensity factor, K

$$K = F(a)*s(\pi a)^{1/2} \quad \ldots\ldots\ldots\ldots\ldots \quad (10)$$

in which s = applied stress; F(a) = finite geometry correction factor which may depend on a; and a = crack depth for a surface flaw or half-width for penetration flaws. The stress intensity factor depends on the crack size, structural geometry, and applied far-field stress.

It has been found from extensive experimental data that the crack growth rate, da/dN, and the stress intensity factor range, δK are related by

$$da/dN = C(\delta K)^{n} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

in which δK is obtained from the previous equation, by replacing s with the applied stress range S, and C and n = experimental constants which depend on such factors as the mean cyclic stress, the test environment, and cyclic frequency.

It should be noted that fracture mechanics may be used to arrive at a relation between stress and cycles to failure. Rearranging Equation 11, and integrating

$$NS^m = \frac{1}{C} \int_{a_i}^{a_f} \frac{da}{[F(a)*(\pi a)^{\frac{1}{2}}]^n} \quad .... (12)$$

In where a_i, a_f = initial and final flaw sizes. Equation 12 may be compared with equation 7.

There is the possibility of retarding or inhibiting the growth of the flaws in concrete structures subjected to cyclic loading by using closely spaced randomly dispersed steel fibers as reinforcement in the concrete or mortars and also a greater fatigue strength observed.

3. PROBLEM STATEMENT AND OBJECTIVES

3.1 Statement of the Problem

In many applications, particularly in pavements, bridge deck overlays and offshore structures, the flexural fatigue strength is an important design parameter because these structures are designed on the basis of fatigue load cycles. However, there is little information available of performance of Corrugated Steel Fiber Reinforced Concrete when subjected to flexural fatigue loading or cyclic loading (50). Therefore there is a need to develop reliable data on flexural fatigue properties, and the fresh and hardened concrete properties of corrugated steel fiber reinforced concretes.

3.2 Research Objectives

The major objectives of the investigation were to determine the properties and to evaluate the performance characteristics of the fiber reinforced concrete subjected to fatigue loading. The other objectives of this investigation were as follows:

1. To study the influence of the addition of fibers on the fresh concrete properties pertaining to workability, finishability, unit weight and air content.
2. To determine the hardened concrete properties such as compressive strength, static flexural strength,

ASTM toughness indices, flexural fatigue strength, and endurance limit.

3. To determine the fatigue strength and endurance limit of fiber reinforced concrete, when subjected to non-reversed fatigue loading.
4. To verify the actual fiber distribution in the fiber concrete and to verify the compression strength actually achieved in the field by core testing.
5. To develop a constitutive model for the prediction of the flexural fatigue life of corrugated steel fiber reinforced concrete using linear regression analysis.

4. MATERIALS, MIXES, AND TEST SPECIMENS

4.1 Materials

The following materials were used in the concrete used for the construction of the pavement at Sheridan Lake Road in Rapid City, S.D. The basic concrte mix proportions are given in Table 5.

Fibers: XOREX I, 50.8 mm (2 inch) long steel fibers conforming to ASTM A820, Type 1 were added at 0.50% by volume 39.16 Kg/cu.mt. (66 lbs/cu.yd.). The equivalent diameter of the fibers was 0.76 mm to 1.27 mm (0.03 to 0.05 inch) with an aspect ratio of 40 to 65.

Cement: Type I/II cement satisfying ASTM C150 was used for all the mixes.

Water: The water used was tap water from the Rapid City Municipal Supply System.

Fly Ash: Fly ash was used as a partial substitute for some of the cement.

Coarse Aggregate: The maximum size of the coarse aggregate used in the mix was 38.1 mm (1 1/2 inch). The results of sieve analysis are given in Table 4A, 4B.

Fine aggregate: consisted of natural sand. Tables 4C, 4D show the results of sieve analysis done for the fine aggregate.

Superplasticizer: The superplasticizer used was Rheobuild 1000 manufactured by Master Builders.

Air Entraining Agent: The air entraining agent used was Micro air, manufactured by Master Builders.

4.2 Mixes

The left side of the Sheridan lake road pavement was placed on October 12, 1990 and the right side on November 15, 1990. All specimens were obtained from the fresh concrete used in the construction of the pavement at the time of construction. The specimens were made in steel moulds. All the specimens were covered with plastic sheets and were placed in an insulated wooden box at the site. After 24 hours, the specimens were taken to the SDSM&T Civil Engineering laboratory and were demolded and placed in lime saturated water for curing. Specimens made on October 12, 1990 were designated as SDMIX 1, and Specimens made on November 15, 1990 were designated as SDMIX 2.

All the testing equipment required for testing the fresh concrete was taken to the site. Placing of the concrete started at 11.00 A.M. The concrete was brought to the site by concrete trucks. Fresh concrete tests were done by a group of graduate students under the supervision of Ramakrishnan.

4.3 The following specimens were made

SDMIX Number 1

1. Six 152mm x 305 mm (6"x12") cylinders
2. Nine 152mm x 152mm x 533 mm (6"x6"x21") beams

SDMIX Number 2

1. Six 152mm x 305mm (6"x12") cylinders
2. Nine 152mm x 152mm x 533mm (6"x6"x21") beams

5. TESTS FOR FRESH AND HARDENED CONCRETE

5.1.1 Fresh concrete tests

The freshly mixed concrete was tested for slump (ASTM C143), air content (ASTM C231), fresh concrete unit weight (ASTM C138), time of flow through inverted cone (ASTM C995), and the temperature of the concrete. These tests are recommended for fiber reinforced concrete by the ACI committee 544.

5.1.2 Inverted cone test

The test for inverted cone time is a new test for the determination of workability of fibrous concrete. The test apparatus consists of a regular slump cone (ASTM C143) and 0.02832 cu.mt. (one cu.ft.) yield bucket (ASTM C29). In addition, an internal vibrator is used (ASTM C192) for compaction of the concrete. The inverted cone is loosely filled with concrete and struck off level at the top. The vibrator is placed at the center of the cone in running condition, then lowered vertically until it hits the bottom of the yield bucket, allowing approximately 2 seconds for this performance. The vibrator should touch the bottom of the yield bucket during the test. The time from initial immersion of the vibrator to when the slump cone is empty is recorded as the test time. Primarily, the test measures the mobility or the fluidity of the mix, and the result is dependent on the many parameters such as aggregate size, shape, gradation, air content, admixtures, volume of fibers and type of fibers.

5.2 TESTS FOR HARDENED CONCRETE

5.2.1 Compressive strength and static modulus

The cylinders were tested for compressive strength at 14 and 28 days, according to ASTM C39. Prior to the compression test the same cylinders were also tested for static modulus of elasticity (ASTM C469) and dry unit weight.

5.2.2 Static flexure test

The beams were tested for static flexural strength (ASTM C1018) at 14 and 28 days. According to ASTM C1018, third point loading was applied to the beams in the static flexure test. Beam specimens of length 152mm x 152mm x 533mm (6"x6"x21") were tested. These were tested over a simply supported span of 457 mm (18 inches). The span to depth ratio of 3 is sufficiently large to obviate the effect of shear stress on the mid-span deflection, and the deflection was measured at the mid-span, by using the dial gauge, accurate to 0.00254 mm (0.0001 inch).

The dial gauge was located at the mid span of the bottom face of the specimen. The static flexure test setup is shown in Figure 11. This test is a deflection controlled test. The rate of deflection was kept in the range of 0.0508 to 0.1016 mm (0.002 to 0.004 inch) per minute as per ASTM C1018. At every 0.00508 mm (0.0002 inch) increment in the deflection, the loads applied were noted-down until the first crack appeared, and later the loads were recorded at different intervals. The load corresponding to first crack formation is called the first crack load. From the test results load-deflection curves were drawn and ASTM toughness indices were calculated. The flexural toughness factor and the equivalent flexural strength were also calculated using Japanese standard method.

5.2.3 Toughness

Toughness is a measure of energy absorption capacity and it is used to characterize steel fiber reinforced concrete's ability to resist fracture when subjected to static, dynamic and impact loads. The United States and Japan are two countries that have specific standards in regard to steel fiber reinforced concrete (51). Load deflection response of standard SFRC beams tested in static flexure is the basis used in several proposed indices to measure the toughness of the material. Both countries use third point loading configuration.

The values obtained from these standards would differ considerably. Both the U.S. and Japanese standards were used. These standards and procedures are outlined below:

1. ASTM standard C 1018 method

The ASTM standard C1018-85 involves the determination of the amount of energy required to deflect a simply supported beam under third point loading, a specific multiple of the first crack deflection. The numbers obtained are called toughness indices. The toughness indices are dimensionless parameters which define the shape of the load deflection curve.

For plain concrete the toughness index is equal to 1, since all plain concrete beams fail immediately after the first crack occurs. The current standard C1018-89 defines toughness as the energy equivalent to the area under the load deflection curve up to a specified deflection. The indices are I_5, which is obtained by dividing the area up to a deflection of 3.0 times the first crack deflection by the area up to the first crack, I_{10} and I_{20} consider the areas up to 5.5 and 10.5 times the first crack deflections respectively. Figure 12 (from ASTM C1018-89) shows the characteristics of load deflection curve. The areas used for calculating the toughness indices are also indicated.

The residual strength factor $R_{5,10}$ is defined as the number obtained by calculating the value of $20(I_{10}-I_5)$, and the residual strength factor $R_{10,20}$ is defined as the number obtained by calculating the value $10(I_{20}-I_{10})$. A residual value of 100 corresponds to perfectly plastic behavior. Lower values indicate, lower plastic performance. Plain concrete has a residual strength factor of zero.

2. Japanese standard JCI-SF4 method

In the Japanese method, the test should be continued until a deflection equal to 1/150 of the span is reached. This deflection is equal to 3.048 mm (0.12 inch) for a 457 mm (18 inch) span length. The deflection thus obtained is greater than what is measured by the ASTM standard method. Japanese flexural toughness factor is calculated by measuring the energy equivalent to the area under the load deflection curve up to 1/150 of the span (3.048 mm) is reached. From the flexural toughness factor, equivalent flexural strength is calculated by using the following formula:

Equivalent flexural strength (JCI) - It is defined by

$$\sigma_b = T_{jci} / \delta_{150} \times s/Wd^2$$

Where σ_b = equivalent flexural strength (jci)

T_{jci} = flexural toughness factor

s = span of the specimen

W = Width of the specimen

d = depth of the specimen

5.2.4 Flexural fatigue test

Fatigue strength is defined as the maximum stress at which the specimen withstands 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.

In the test for flexural fatigue, third point loading was used with a span of 457 mm (18 inches). In this test, the lower load limit was set at 10% of the average maximum load obtained from the static flexure test. For the first beam in each mix the upper load limit was set at 80% of average maximum flexural load for the set. The fatigue test was run between these limits. If the beam failed before completing 2 million cycles, the upper limit was reduced for the next specimen. If the beam survived, another beam was tested at the same upper load as a replicate.

The frequency of loading used was 20 cycles per second (Hz) for all the tests. The Material Test System (MTS) machine was used for all the tests. It has a cyclic load capacity of 55,000 pounds. The monitor system consists of a MTS 436 control unit, a Hewlett packard oscilloscope, and a digital multimeter working with MTS load cell. The machine could be operated in any of the three modes: load control (force applied to the specimen) or 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 because it is closely related to the true cyclic loading behavior.

There was also a counter 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 cutoff switch was also provided which could turn off the machine when the beam breaks. The fatigue test setup is shown on Figure 13.

6. TEST RESULTS, ANALYSIS, AND DISCUSSIONS

6.1.1 Fresh concrete properties

The results of the tests on fresh concrete are given in Tables 3A through 3E. For all mixes atmospheric temperature, humidity, and concrete temperature were recorded to ensure that all the mixes were done under approximately similar conditions. The temperature and humidity varied in the range of 48° to 65° F, and 35% to 40% respectively. The concrete temperature range was 59° to 70° F.

6.1.2 Workability

A tendency towards balling or formation of fiber nodules is a serious problem in fiber reinforced concrete. Balling reduces workability and increases segregation. Two tests were done to determine the workability of the mixes, namely slump, and inverted cone time. These test results indicate that satisfactory workability can be maintained by using steel fibers 0.5 by volume 39.16 Kg/cu.mt. (66 lbs/cu.ft.). This was achieved by adjusting the amount of superplasticizer used.

The results indicated that inverted cone time and slump values were inversely proportional (Fig.14). The fresh concrete properties are given in Tables 3A through 3E.

6.1.2 Fresh concrete unit weight and fiber content

The results of fresh concrete unit weight are given in Table 3A. The average fresh concrete unit weight was 2411 Kg/m^3 (150.53 lbs/cu.ft.) It was observed that the unit weight decreased considerably with increase in air content.

Fiber content in each mix was determined by collecting all the fibers from the concrete used in the air content test. Similarly the fiber content was also determined by collecting all the fibers from the concrete used in the unit weight test. The results are given in Tables 3C, 3D, and 3E. The results indicate that the fiber content was almost uniform throughout the concrete.

6.2 Hardened concrete properties

6.2.1 Compressive strength

The compressive strength results for 14 and 28 days are given in Table 6. The average compressive strength was 35.23 MPa (5110 psi) at 14 days and 39.23 MPa (5690 psi) at 28 days. It was observed that the compressive strength had increased by 12 % from 14 to 28 days. Figure 16 shows increase in compressive strength from 14 to 28 days. It was observed that compressive strength decreased with an increase in air content, as shown in Fig.17. It was also observed that fibers do not significantly influence the compressive strength of the concrete.

The values of within-test standard deviation and coefficient of variation values for all the mixes are given in Table 6. The values of coefficient of variation was below 4 which is the permissible limits for the research work.

An important observation during the compression tests was that the fiber concrete cylinders failed in a ductile mode. The plain concrete cylinders failed instantly and shattered into pieces with a loud noise whereas the fiber concrete cylinders did not fail suddenly, and they did not shatter at failure. The strengthening mechanism of the fibers involves transfer of stresses from the matrix to the fiber by interfacial shear, or by interlock between the fiber and matrix if the fiber surface is deformed. Stress is thus shared by the fiber and matrix until the matrix cracks, and total stress is progressively transferred to the fibers.

6.2.2 Static modulus

The test results for static modulus are given in Table 6. The average static modulus was 36200 MPa ($5.25x10^6$ psi) and its range was from 34200 to 38000 MPa ($4.96x10^6$ to $5.51x10^6$ psi). The static modulus was proportional to the compressive strength of the concrete and the addition of fibers did not influence the static modulus.

6.2.3 Dry unit weight

Table 6 gives the values of dry unit weight. The average dry unit weight of hardened concrete was 2380 Kg/m^3 (148.6 lbs/cu.ft.) with a minimum 2366 Kg/m^3 (147.7 lbs/cu.ft.) and maximum 2400 kg/m^3 (149.8 lbs/cu.ft.). Unit weight slightly increased due to the fiber content, but there was not any significant difference in the fresh and hardened concrete unit weight at 28 days. Figure 18 shows the variation of unit weight of dry and fresh concretes.

6.2.4 Static flexural strength

Static flexural strength results for 14 and 28 days are given in Table 7. Due to the addition of steel fibers the ultimate load carrying capacity of concrete is improved as compared with normal concrete. Modulus of rupture values at first crack formation for fiber reinforced concrete are higher than that of plain concrete. The mode of failure was a simultaneous yielding of the fibers and the failure of the matrix. It seems obvious that the deformations (corrugations) in the fibers had contributed significantly to the increase in bond between fibers and the matrix. The significance of strong bond can be seen from the load deflection curves.

6.2.5 Load-deflection behavior

Load deflection curves are a standardized method of quantifying the energy a beam absorbs during its load induced flexural deflection. The area under the curve represents the energy absorbed by the beam.

Load deflection curves were drawn using the data from the static flexure test. These curves indicate that the load carrying capacity of the concrete had increased due to the addition of fibers, whereas in plain concrete, the beam fails immediately after the appearance of the first crack, and the flexural rigidity of the beam had also increased, giving lower deflection at the corresponding load level when compared to that of plain concrete beams. The rate of degeneration of moment of inertia is slowed down by the fibers by trying to resist the propagation of the crack growth. Load deflection curves are shown in Fig. 19 to 27. Load deflection curves will show the ehhanced elastoplastic behavior due to the addition of fibers.

6.2.6 Post-crack load drop phenomenon

The difference between the maximum load and the load recorded at a deflection equal to three times the deflection measured at first crack is defined as the post-crack load drop. The post-crack load drop phenomenon is lesser with increase in fiber content.

6.2.7 Toughness indices

The Toughness index (ASTM C1018) is a dimensionless parameter which fingerprints the shape of the load-deflection curve. By including the percentage load-drop values as suggested previously, the fingerprinting of the shape of load deflection curve can be further improved. Toughness indices have been explained on the basis of three service levels, identified as multiples of the first crack deflection. The toughness indices for fiber concrete vary greatly depending on the position of the crack, the type of fiber , aspect ratio, volume fraction of the fiber and the distribution of the fibers.

The calculated values of ASTM toughness indices I_5, I_{10}, and I_{20} are tabulated in Table 8. The ratios of I_{10}/I_5 and I_{20}/I_{10} are very good indicators of plastic behavior of that particular specimen. Table 8A shows Japanese toughness indices and equivalent flexural strength.

6.2.8 Flexural fatigue behavior

One of the primary objectives of this investigation was the study of the flexural fatigue behavior of the concrete to be used in the construction of the pavement. The beams of curves on an S-N diagram.

The additional fatigue data used in the statistical analysis were collected from three Master of Science theses from The South Dakota School of Mines and Technology (44,45, 46). For all the tests the same mix proportions, fiber type, and fiber volume were used. All the specimens were subjected to flexural fatigue with third point loading at a frequency of 20 cycles per second. A data file was made, with 35 observations for further statistical analysis.

6.3.2 Assumptions made for analysis

In the regression analysis fatigue stress was taken as dependent or response variable on the Y-axis and number of cycles as independent or predictor variable on the X-axis. The independent variable was assumed to be error free for better prediction of fatigue stress. Fatigue response of concrete can be better predicted by dividing into a low-cycle fatigue and high-cycle fatigue. The range of low-cycle fatigue is from one to 1000 cycles and the range of high-cycle fatigue is from 1000 to 2 million cycles. It was suggested in an American Society For Testing and Materials (ASTM) publication (27) that the fatigue life N is assumed to be normally distributed. The given data tested for normal distribution by using a software **SIGMAPLOT50.** The probability plot indicates that fatigue data is normally distributed between 30,000 and 2,000,000 cycles.

As proposed by Ramakrishnan (16), fatigue stress f_{max} is defined as the maximum flexural stress at which the beam

6.3.6 Developed Model

This research has provided the following flexural fatigue model and S-N curve that will serve as a guide for design of corrugated steel fiber reinforced concrete subjected to cyclic loading.

$$f_{fmax} = 5.6370 - 0.143521 * \ln N$$

$$f_{max}/f_r = 1.00227 - 0.026247 * \ln N$$

Through the developed model flexural fatigue stress can be predicted reasonably well for a structure designed to withstand the required number of load cycles.

6.4 Core tests

The steel fiber reinforced concrete pavement was placed on two different dates: the left side on October 12, 1990 and the right side on November 15, 1990. The SFRC pavement sections have two thicknesses 139.7 and 152.4 mm (5.5 and 6 inch), whereas the plain concrete pavement has 190.5 mm (7.5 inch) thickness. Cores were taken to verify the thickness of SFRC Pavement, to check for the consolidation and fiber distribution, and to determine the compressive strengths.

Cores were taken from both sides of the pavements. Four cores were taken on the left side of SFRC Pavement at distances of 55.78 meters and 67 meters from the starting point of the SFRC pavement which was placed on October 12, 1990. Four more cores were taken on the right side of SFRC pavement at distances of 64.9 meters and 88 meters from the

7. CONCLUSIONS

The following conclusions have been drawn from the field observations, the experimental and the analytical investigation carried out on steel fiber reinforced concrete used for the construction of the pavement at the Sheridan lake road in Rapid city, South Dakota.

1. No balling or tangling of fibers occurred during mixing and placing of steel fibers and handling of the fiber mix with 0.50% fiber content (39.16 Kg/m^3). Good workability was achieved by addition of a suitable amount of superplasticizer.
2. Steel fibers improved the toughness and ductility of concrete. The mode of failure was changed from brittle failure to a partial ductile failure with a consistent increase in the post-crack energy absorption capacity. Significant good bonding between the fibers and matrix can be seen from the load deflection curves.
3. Fatigue strength and endurance limit expressed as a percentage of its modulus of rupture had higher values due to addition of steel fibers compared to that of plain concrete.
4. The flexural fatigue strength increased considerably with addition of steel fibers.
5. With the use of fiber reinforced concrete, the thickness of the pavement can be considerably

reduced.

6. The core test results showed that the fiber reinforced concrete in place had much higher compressive strength values than the specified strength, the fiber distribution was uniform, and the fiber reinforced concrete was well compacted without any voids.
7. The fiber reinforced concrete reached an endurance limit at two million cycles, as seen from models developed.
8. The after-fatigue static flexural strength was greater than the pre-fatigue static flexural strength.
9. Dividing the spectrum of fatigue life into low-cycle and high-cycle regions has improved the prediction of fatigue data.
10. From the developed model, endurance limit for the 0.5 percent volume of steel fiber reinforced concrete predicted is 3.56 MPa (516 psi).

8. RECOMMENDATIONS FOR FURTHER STUDY

Additional research is recommended in the following areas:

1. Additional experimental study is required to determine the optimum fiber content to achieve high performance and workability.
2. Bonding strength between concrete and fiber should be improved by using different geometry of fibers and also by adding some admixtures such as silica fume, flyash, etc.
3. The influence of more random variation of the stress levels on fatigue strength.
4. Strain in concrete should be monitored under fatigue testing on order to study the performance with respect to deformations in various cyclic ranges.

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C-138-81^{e1} Standard test method for unit weight, yield, and air content (Gravimetric) of concrete.
C-143-89a Standard test method for slump of hydraulic cement concrete.
C-231-89a Standard test method for air content of fiber mixed concrete by the pressure method.
C-469-87a Standard test method for elastic modulus of elasticity and poisson's ratio of concrete in compression.
C-1018-89 Standard test method for flexural toughness and first crack strength of fiber reinforced concrete (Using third point loading).
E-380-89a Standard practice for use of the international system of units (SI) (the modernized metric system).

TABLE 1. FATIGUE LOAD SPECTRUM

Low-cycle Fatigue	High-Cycle Fatigue		Super-High cycle Fatigue	
Structures Subjected to Earthquake	Airport pavements & Bridges	Highway and railway Bridges Highway pavements, and concrete railroad ties	Mass rapid Transit structures	Sea structures

0 10^1 10^2 10^3 10^4 10^5 10^6 10^7 10^8 $5X10^8$

Number of cycles

TABLE 2-PARAMETERS AFFECTING FATIGUE PERFORMANCE OF CONCRETE

Concrete composition	Effect of environment	Loading conditions	Mechanical properties
Air content	Temperature	Reversible loading	Compressive strength
w/c ratio	Moisture content	Variable stress	Tensile strength
Aggregate type	Aggressive agents	Constant stress	Elastic modulus
Cement content	Corrosion	Stress range	Modulus of fracture
Pozzolans	Immersion	load-wave forms	Prestress
Age, Curing conditions		Loading rate	Steel fibers
		Load amplitude	Precracking

TABLE 3A. PROPERTIES OF FRESH CONCRETE

MIX #	FIELD TEMP. (^{o} C)	FIELD HUMIDITY %	CONCRETE TEMP. (^{o} C)	UNIT WEIGHT (Kg/m^{3})
SDMIX #1A	18.3	35	16.6	2390.18
SDMIX #1B	21.6	26	21.1	2409.41
SDMIX #2	12.7	35	15.0	2433.44

TABLE 3B. PROPERTIES OF FRESH CONCRETE

MIX #	SLUMP (mm)	AIR CONTENT (%)	INVERTED CONE TIME (Sec.)
SDMIX #1A	50.80	6.0	12.49
SDMIX #1B	40.64	5.3	23.74
SDMIX #2	25.40	6.0	33.00

TABLE 3C. DETERMINATION OF FIBER CONTENT IN THE CONCRETE SDMIX # 1 A

Concrete taken from Air meter container:

Weight of container and fibers	= 1.36 Kgs (2.998 lbs)
Weight of container	= 1.083 Kgs (2.386 lbs)
Weight of fibers	= 0.277 Kgs (0.6105lbs)
Volume of air container	= 0.00719 cu.meter (0.254 cu.ft.)
Fiber content	= 38.50 Kg/cu.meter (64.88 lbs/cu.yd.)

Concrete taken from Unit weight container:

Weight of container and fibers	= 1.331 Kgs (2.934 lbs)
Weight of container	= 1.071 Kgs (2.36 lbs)
Volume of container	= 0.006835 cu.meter
Weight of fibers	= 0.260 Kgs (0.574 lbs)
Fiber content	= 38.56 Kg/cu.meter (64.99 lbs/cu.yd.)

TABLE 3D. DETERMINATION OF FIBER CONTENT IN THE FIELD CONCRETE SDMIX # 1 B

Concrete taken from Air meter Container:

Weight of fibers and container	= 1.339 Kgs (2.952 lbs)
Weight of container	= 1.082 Kgs (2.384 lbs)
Weight of fibers	= 0.257 Kgs (0.566 lbs)
Fiber content	= 35.72 Kg/cu.meter (60.21 lbs/cu.yd.)

Concrete taken from Unit Weight container:

Weight of fibers and container	= 1.290 Kgs (2.843 lbs)
Weight of container	= 1.075 Kgs (2.369 lbs)
Weight of fibers	= 0.215 Kgs (0.473 lbs)
Fiber content	= 31.89 Kg/cu.meter (53.75 lbs/cu.yd.)

TABLE 3.E. DETERMINATION OF FIBER CONTENT IN THE CONCRETE SDMIX # 2

Concrete taken from Air meter container:

Weight of fibers and container	= 1.337 Kgs (2.946 lbs)
Weight of container	= 1.083 Kgs (2.386 lbs)
Weight of fibers	= 0.252 Kgs (0.555 lbs)
Volume of air container	= 0.00719 cu.meter (0.254 cu.ft.)
Fiber content	= 34.99 Kgs/cu.meter (2.1846 lbs/cu.ft.)

Concrete taken from Unit weight measure container:

Weight of fibers and container	= 1.334 Kgs (2.369 lbs)
Weight of container	= 1.075 Kgs (2.369 lbs)
Weight of fibers	= 0.259 Kgs (0.570 lbs)
Volume of container	= 0.006835 cu.meter (0.2414 cu.ft.)
Fiber content	= 37.87 Kg/cu.meter (63.83 lbs/cu.yd.)

TABLE 4A. SIEVE ANALYSIS - COARSE AGGREGATE
(3/4" maximum size)
(Crushed Limestone)

Sieve Size Passing through	retained on	Percent Retained by weight
1.5 in	1 in	0
1.0 in	3/4 in	2.91
3/4 in	1/2 in	45.72
1/2 in	3/8 in	30.33
3/8 in	1/4 in	14.15
1/4 in	# 4	3.90
# 4	# 8	1.19
# 8	Pan	1.74

Fineness Modulus = 6.80
Absorption Coefficient = 0.54 %

TABLE 4B. SIEVE ANALYSIS FOR COARSE AGGREGATE USED
For 60 percent 1" max. and 40 percent 3/8" max. size

Sieve Size Passing through	Retained On	Cumulative Percent Retained	ASTM Grading Requirement for size #67 Weight Percent
1.0 in	3/4 in	2.91	100
3/4 in	1/2 in	45.72	90-100
1/2 in	3/8 in	30.33	-
3/8 in	1/4 in	14.15	20-55
1/4 in	# 4	3.90	-
# 4	# 8	1.19	0-5
# 8	Pan	1.74	-

Fineness Modulus = 6.59
Absorption Coefficient = 0.54%

TABLE 4C. SIEVE ANALYSIS FOR FINE AGGREGATE USED
(Natural fine aggregate)

Sieve Size Passing through	Sieve Size Retained On	Percent Retained by weight
1/4 in	#4	0.10
#4	#8	11.32
#8	#16	23.70
#16	#30	31.44
#30	#50	19.39
#50	#100	10.17
#100	#200	3.36
#200	Pan	0.52

Fineness Modulus = 295.27/100 = 2.95
Saturated Surface dry specific gravity = 2.63
Absorption Coefficient = 1.64 %

TABLE 4D. SIEVE ANALYSIS FOR FINE AGGREGATE USED
(San Gabriel river fine aggregate)

Sieve Size Passing through	Sieve Size Retained On	Percent Retained by weight
1/4 in	#4	4.38
#4	#8	19.45
#8	#16	15.92
#16	#30	19.27
#30	#50	21.62
#50	#100	13.63
#100	#200	4.17
#200	Pan	1.53

Fineness modulus = 3.02
Absorption Coefficient = 1.64%

TABLE 5. BASIC MIX PROPORTIONS

Fine Aggregate	930.93 Kg/cu.mt.
Coarse Aggregate	925.60 Kg/cu.mt.
Cement	390.41 Kg/cu.mt.
Water	156.03 Kg/cu.mt
Superplasticizer	See note.
Air-entraining Agent	0.3284 Kg/cu.mt.
Fibers	39.16 Kg/cu.mt.

Notes:

1. Superplasticizer was adjusted to get required workability.
2. W/C ratio by weight = 0.40
3. Type of steel used : 50.8 mm long corrugated carbon steel fibers.

TABLE 6. CYLINDER COMPRESSIVE STRENGTH

SP.#	AGE (DAYS)	DIMENSIONS LENGTH (mm)	DIA. (mm)	UNIT Wt. (Kgs/m^3)	STATIC MODULUS (MPa)*10^3	COMPRESSIVE STRENGTH (MPa)
SDSPC-4	14	304.80	152.15	2366.15	36.62	35.89
SDSPC-5	14	306.07	152.15	2367.75	36.82	34.68
SDSPC-6	14	308.86	152.91	2390.18	36.82	34.27
MEAN				2374.68	36.69	34.95
S.D				13.436	0.115	0.842
C.V				0.006	0.003	0.024
SDSPC-1	28	304.80	152.65	2399.79	36.47	37.92
SDSPC-2	28	304.80	152.15	2375.76	36.75	41.37
SDSPC-3	28	305.05	152.15	2367.75	29.30	38.34
MEAN				2381.10	34.17	39.21
S.D				16.674	4.223	1.883
C.V				0.007	0.124	0.048
SDSPC-8	14	305.05	152.65	2356.54	37.99	45.02
SDSPB-9	14	304.80	152.15	2377.37	36.75	38.68
SDSPB-10	14	304.80	152.65	2364.55	36.54	41.44
MEAN				2366.15	37.09	41.72
S.D				10.507	0.784	3.179
C.V				0.004	0.021	0.076
SDSPB-7	28	304.80	151.64	2390.18	36.96	47.23
SDSPB-11	28	304.80	151.89	2382.17	36.89	49.16
SDSPB-12	28	304.80	152.15	2374.16	36.75	47.02
MEAN				2382.17	36.87	47.81
S.D				8.009	0.107	1.179
C.V				0.003	0.003	0.025

S.D. : Standard Deviation
C.V. : Coefficient of Variation

TABLE 7. FIRST CRACK STRENGTH AND MAXIMUM FLEXURAL STRENGTH

SP #	AGE (days)	FOR THE FIRST CRACK LOAD (Newtons)	FOR THE FIRST CRACK DEFLECTION (mm)	MAX. LOAD (Newtons)	FLEXURAL STRESS (MPa)
SDSPB-2	14	32025.0	-	32025.0	3.93
SDSPB-3	14	37363.0	0.0305	37363.0	4.52
SDSPB-5	14	31136.0	0.0457	31136.0	3.83
MEAN		33508.0	0.0381	33508.0	4.09
S.D.		3367.9	0.0108	3367.9	0.37
C.V.		0.1	0.2828	0.1	0.09
SDSPB-6	28	45369.0	0.0203	45369.0	5.55
SDSPB-7	28	45369.0	0.0356	45369.0	5.58
SDSPB-9	28	44480.0	0.0254	44480.0	5.41
MEAN		45072.0	0.0270	45072.0	5.51
S.D.		513.3	0.0078	513.3	0.09
C.V.		0.01	0.2864	0.01	0.02
SDSPB-10	14	45814.0	0.0203	45814.0	5.48
SDSPB-15	14	44924.0	0.0254	44924.0	5.45
MEAN		45369.0	0.0229	45369.0	5.47
S.D.		629.3	0.0036	629.3	0.02
C.V.		0.01	0.1571	0.01	0.00
SDSPB-11	28	46259.0	0.0508	46259.0	5.55
SDSPB-12	28	47148.0	0.0406	47148.0	5.69
MEAN		46703.5	0.04572	46703.5	5.62
S.D.		628.6	0.00718	628.6	0.09
C.V.		0.01	0.1571	0.01	0.02

S.D. - Standard Deviation
C.V. - Coefficient of Variation

TABLE 8. TOUGHNESS INDICES OF FIBER REINFORCED CONCRETE BEAMS

SP.#	AGE (days)	FLEXURAL STRENGTH (MPa)	FIRST CRACK TOUGHNESS (N-Metre)	I_5	I_{10}	I_{20}	I_{10}/I_5	I_{20}/I_{10}
SDSPB-5	14	3.93	0.712	3.63	6.19	11.05	1.70	1.78
SDSPB-3	14	4.52	0.569	4.76	9.27	16.78	1.94	1.80
MEAN		4.23	0.641	4.19	7.73	13.92	1.82	1.79
S.D		0.42	0.101	0.79	2.17	4.05	0.17	0.02
C.V		0.09	0.158	0.19	0.28	0.29	0.09	0.01
SDSPB-6	28	5.55	0.461	2.88	4.57	8.95	1.58	1.95
SDSPB-7	28	5.58	0.807	2.04	3.43	6.01	1.68	1.75
SDSPB-9	28	5.41	0.565	2.69	5.86	8.92	2.17	1.52
MEAN		5.52	0.611	2.54	4.62	7.96	1.81	1.74
S.D		0.09	0.177	0.44	1.22	1.68	0.31	0.22
C.V		0.02	0.290	0.17	0.26	0.21	0.17	0.12
SDSPB-10	14	5.48	0.465	2.95	5.00	8.63	1.69	1.72
SDSPB-15	14	5.45	0.571	3.03	5.24	7.10	1.72	1.35
MEAN		5.47	0.518	2.99	5.12	7.87	1.71	1.54
S.D		0.02	0.075	0.06	0.17	1.08	0.02	0.26
C.V		0.00	0.145	0.02	0.03	0.14	0.01	0.17
SDSPB-11	28	5.55	1.175	3.90	6.43	9.78	1.65	1.52
SDSPB-12	28	5.69	0.958	4.83	8.50	13.95	1.76	1.64
MEAN		5.62	1.067	4.37	7.47	11.87	1.71	1.58
S.D		0.09	0.153	0.66	1.46	2.95	0.08	0.08
C.V		0.02	0.144	0.15	0.19	0.25	0.05	0.05

S.D. - Standard Deviation
C.V. - Coefficient of Variation

TABLE 8A. JAPANESE STANDARD - TOUGHNESS AND EQUIVALENT FLEXURAL STRENGTH

SP.#	AGE (days)	TOUGHNESS (N-meter)	EQUIVALENT FLEXURAL STRENGTH (MPa)
SDSPB-5	14	56.46	2.29
SDSPB-3	14	55.69	2.20
MEAN		56.08	2.24
S.D.		0.54	0.06
C.V.		0.01	0.03
SDSPB-6	28	74.72	2.98
SDSPB-7	28	43.72	1.77
SDSPB-9	28	39.73	1.58
MEAN		52.72	1.68
S.D.		19.15	0.13
C.V.		0.36	0.03
SDSPB-10	14	43.00	1.69
SDSPB-15	14	37.03	1.47
MEAN		40.02	1.58
S.D.		4.22	0.16
C.V.		0.11	0.09
SDSPB-11	28	50.59	1.99
SDSPB-12	28	52.52	2.06
MEAN		51.50	2.03
S.D.		1.29	0.05
C.V.		0.03	0.02

S.D. - Standard Deviation
C.V. - Coefficient of Variation

TABLE 9. FATIGUE TEST RESULTS

SPECIMEN #	DATE OF TESTING	AGE DAYS	DIMENSIONS AT FAILURE SECTION		DYNAMIC LOAD	
					MAX.	MIN.
			WIDTH (mm)	DEPTH (mm)	Newtons	Newtons
SDSPB-1	2-27-91	138	151.59	154.74	30112.9	4301.2
SDSPB-4	2-23-91	134	154.94	156.26	34414.2	4301.2
SDSPB-5	2-24-91	135	155.55	155.04	32265.8	4301.2
SDSPB-13	2-22-91	99	158.80	155.68	30112.9	4301.2
SDSPB-14	2-27-91	104	155.17	153.39	30112.9	4301.2
SDSPB-16	2-21-91	98	156.49	156.03	25811.7	4301.2
SDSPB-17	2-23-91	100	153.16	155.24	32265.8	4301.2

TABLE 9A. FATIGUE TEST RESULTS

SPE. #	MAX. STRESS (MPa)	MIN. STRESS (MPa)	STRESS RANGE (MPa)	f_{max}/f_r	CYCLES AT FAILURE
SDSPB-4	4.158	0.517	3.641	0.748	8200
SDSPB-17	3.999	0.531	3.468	0.719	1000
SDSPB-5	3.944	0.524	3.420	0.709	52200
SDSPB-1	3.792	0.545	3.247	0.682	2000900 +
SDSPB-18	3.792	0.545	3.247	0.682	2000900 +
SDSPB-14	3.772	0.538	3.234	0.679	625000
SDSPB-13	3.579	0.510	3.069	0.643	2000000 +
SDSPB-16	3.096	0.517	2.579	0.557	2000890 +

Average flexural strength f_r = 5.557 MPa (from Table No.4)

+ Beams crossed 2 million cycles and later they were tested for an additional 10 million cycles.

TABLE 10A. ADDITIONAL FATIGUE TESTING OF 10 MILLION CYCLES AFTER THE SPECIMENS DID NOT FAIL AT 2 MILLION CYCLES

SPE.#	MAX. STRESS (MPa)	MIN. STRESS (MPa)	STRESS RANGE (MPa)	f_{max}/f_r	CYCLES AT FAILURE
SDSPB-1	3.792	0.545	3.247	0.682	13600000 *
SDSPB-18	3.792	0.545	3.247	0.682	2186900
SDSPB-13	3.578	0.510	3.068	0.643	10000000 *
SDSPB-16	3.096	0.517	2.579	0.557	10000000 *

* Beams crossed 12 million cycles and later tested for additional 10 million cycles.

TABLE 10B. FATIGUE TESTING OF ADDITIONAL 10 MILLION CYCLES AFTER THE SPECIMENS DID NOT FAIL AT 12 MILLION CYCLES

SPE.#	MAX. STRESS (MPa)	MIN. STRESS (MPa)	STRESS RANGE (MPa)	f_{max}/f_r	TOTAL NO. OF CYCLES
SDSPB-1	3.792	0.547	3.247	0.682	19466300
SDSPB-13	3.578	0.510	3.068	0.643	22009000 +
SDSPB-16	3.096	0.517	2.579	0.557	22008900 +

+ Beams crossed 22 million cycles and later tested for Static flexure test.

TABLE 11. STATIC FLEXURE TEST AFTER FATIGUE

SPEC. #	DATE OF TESTING	DIMENSIONS AT FAILURE SECTION		MODULUS OF RUPTURE
		WIDTH (mm)	DEPTH (mm)	(MPa)
SDSPB-13	4-19-91	154.94	149.86	6.17
SDSPB-16	4-19-91	152.40	149.86	6.27

TABLE 12A. REGRESSION ANALYSIS

Concrete with 0.5% Corrugated steel fibers.

Data: Appendix I

Number of observations = 35

Confidence limits = 95%

Parameter	Estimate	Standard Error	T Value	Prob. Level
Intercept	5.637100	0.287890	19.5808	0.0000
Slope	-0.143521	0.022886	-6.27117	0.0000

Analysis of Variance :

Source	Sum of Squares	Degrees of Freedom	Mean Square	F-Ratio	Prob. Level
Model	4.602119	1	4.602119	39.328	0.0000
Error	3.861668	33	0.117020		
Total (Corr.)	8.463787	34			

Correlation Coefficient = -0.737389   R-Squared = 54.37 %

Stand. Error of Est. = 0.342082

TABLE 12B. REGRESSION ANALYSIS

Concrete with 0.5% Corrugated steel fibers.

Data: Appendix I

Number of observations = 35

Confidence limits = 95%

Parameter	Estimate	Standard Error	T Value	Prob. Level
Intercept	1.002270	0.074633	13.4293	0.00000
Slope	-0.026247	5.93298E-3	-4.42392	0.00010

Analysis of Variance :

Source	Sum of Squares	Degrees of freedom	Mean Square	F-Ratio	Prob. Level
Model	0.153917	1	0.153917	19.57111	0.00010
Error	0.259528	33	0.007864		
Total (Corr.)	0.413445	34			

Correlation Coefficient = -0.610146 R-Squared = 37.23 %

Stand. Error of Est. = 0.088682

TABLE 13A. TEST RESULTS OF SFRC PAVEMENT CORES

TESTED AS PER ASTM C42-87

Specimen #	Dimensions (in mm)				Remarks
	Before Trimming		After Trimming		
	Length	Dia.	Length	Dia.	
1) STA5+50 6'RTφ M1771(3) PENN	154.94	101.60	152.40	101.60	No voids, no honeycombs were found in the specimen. Compaction and fiber distribution were good.
2) STA5+50 6'LTφ M1771(3) PENN	158.75	101.60	154.94	101.60	No voids, no honeycombs were found in the specimen. Compaction and fiber distribution were good.
3) 4+62 6'RTφ M1771(3) PENN	159.00	101.60	157.48	101.60	No voids, no honeycombs were found in the specimen. Compaction and fiber distribution were good.
4) 4+62 6'LTφ 1771(3) PENN	163.07	101.60	162.56	101.60	No voids, no honeycombs were found in the specimen. Compaction and fiber distribution were good.

TABLE 13B. TEST RESULTS OF SFRC PAVEMENT CORES

TESTED AS PER ASTM C42-87

Specimen #	Dimensions (in mm)				Remarks
	Before Trimming		After Trimming		
	Length	Dia.	Length	Dia.	
1) M1771(3) PENN NB4+90 7'LTϕ	153.67	101.60	152.40	101.60	No voids, no honeycombs were found in the specimen. Compaction and fiber distribution were good.
2) M1771(3) PENN NB4+90 7'RTϕ	166.62	101.60	165.10	101.60	No voids, no honeycombs were found in the specimen. Compaction and fiber distribution were good.
3) M1771(3) PENN NB5+55 7'RTϕ	184.66	101.60	180.34	101.60	No voids, no honeycombs were found in the specimen. Compaction and fiber distribution were good.
4) M1771(3) PENN NB5+55 7'LTϕ	159.51	101.60	157.48	101.60	No voids, no honey combs were found in the specimen. Compaction and fiber distribution were good.

TABLE 14A. CORE COMPRESSIVE STRENGTH OF SFRC PAVEMENT

TESTED AS PER ASTM C42-87

DATE OF TESTING : 01/18/1991

Specimen #	Age days	Dimensions		Leng./ dia ratio	Max-imum Load Newt.	Strength Correcti-on factor	Compr-essive Stren-gth (MPa)
		Leng. (mm)	Dia. (mm)				
STA5+50 6'RTϕ M1771(3) PENN	98	152.4	101.6	1.50	315808	0.97	37.78
4+62 6'RTϕ M1771(3) PENN	98	157.5	101.6	1.55	313584	0.97	37.50
STA5+50 6'LTϕ M1771(3) PENN	98	154.9	101.6	1.52	237968	0.97	28.48
4+62 6'LTϕ 1771(3) PENN	98	162.6	101.6	1.60	264656	0.97	31.72

TABLE 14B. CORE COMPRESSIVE STRENGTH OF SFRC PAVEMENT

TESTED AS PER ASTM C42-87

DATE OF TESTING : 01/18/1991

Specimen #	Age days	Dimensions		Length dia ratio	Maximum Load Newt.	Strength Correction factor	Compressive Strength (MPa)
		Leng. (mm)	Dia (mm)				
M1771(3) PENN NB4+90 7'LTφ	64	152.4	101.6	1.50	320256	0.97	34.34
M1771(3) PENN NB4+90 7'RTφ	64	165.1	101.6	1.63	346944	0.98	41.99
M1771(3) PENN NB5+55 7'RTφ	64	180.3	101.6	1.78	342496	0.99	41.85
M1771(3) PENN NB5+55 7'LTφ	64	157.5	101.6	1.55	237968	0.97	28.48

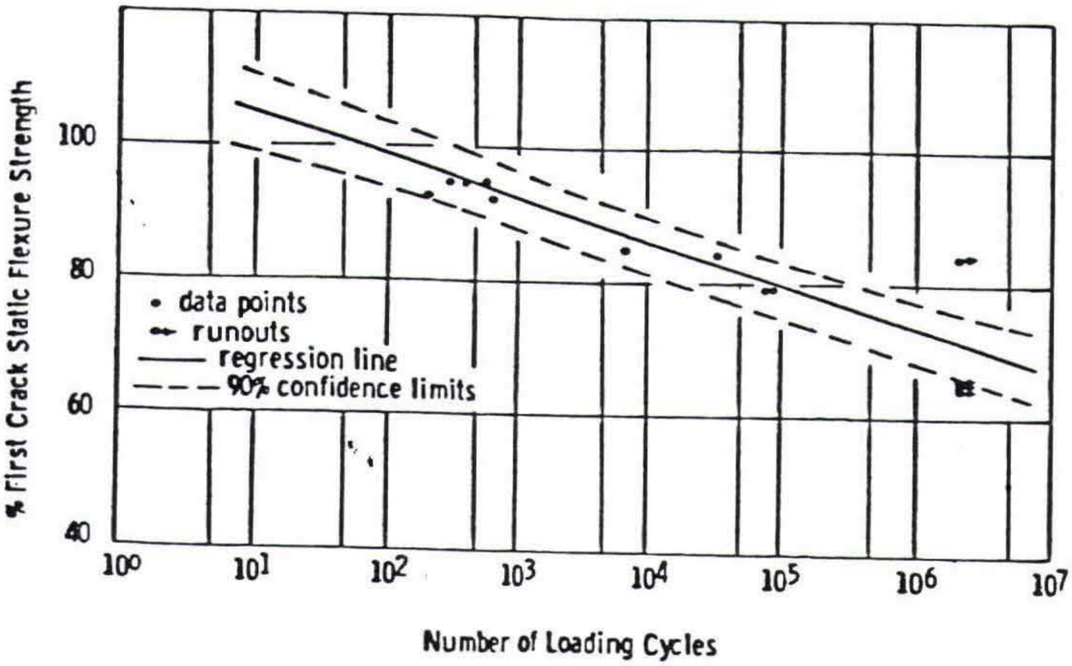


Fig.1 - S-N DIAGRAM FOR FIBER REINFORCED CONCRETE (37)

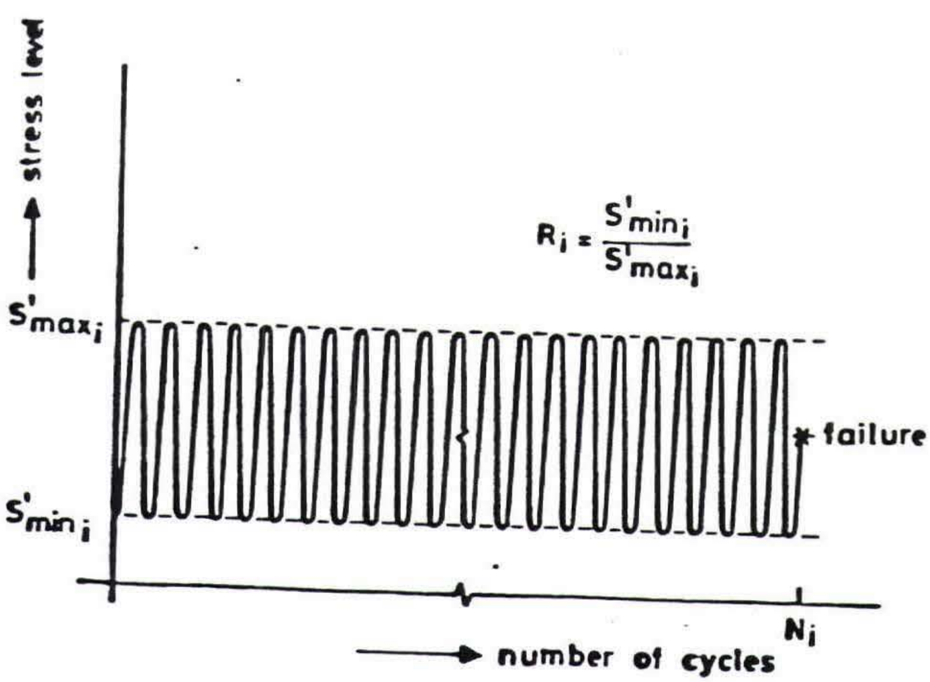


Fig.2 - CONSTANT-AMPLITUDE TEST (36)

FLEXURAL STRESS Vs NUMBER OF CYCLES

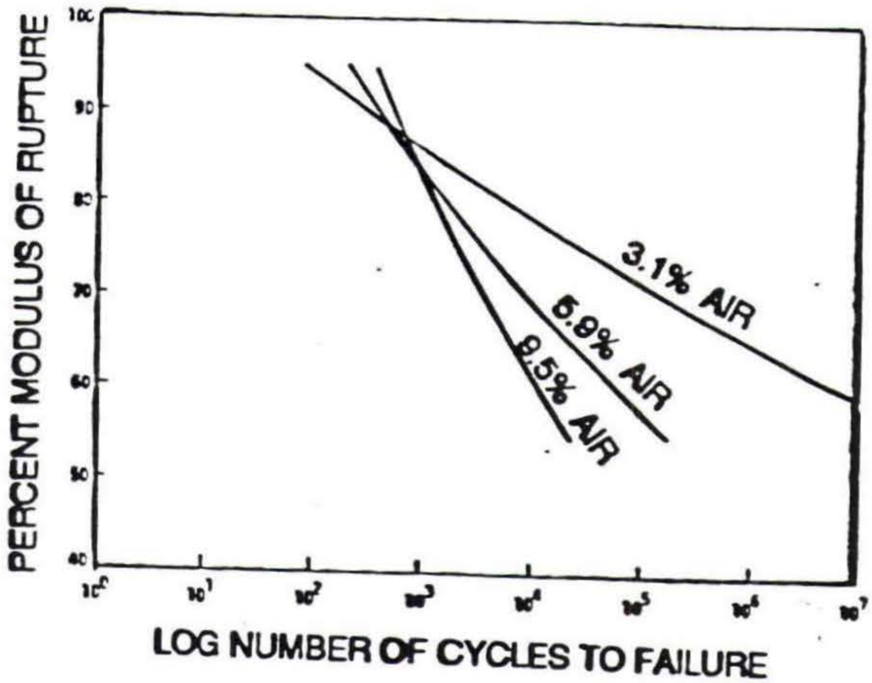


Fig. 3 - EFFECT OF AIR CONTENT ON FATIGUE LIFE (30)

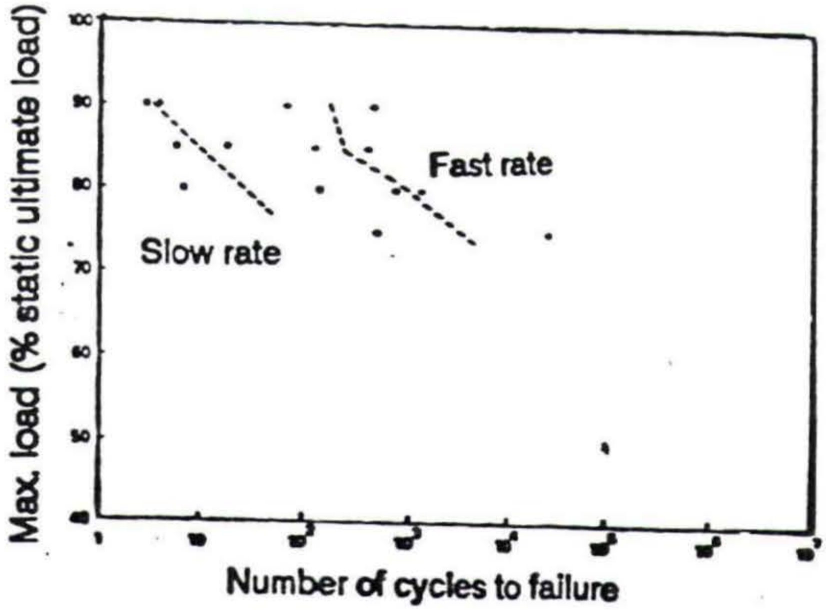


Fig.4 - S-N CURVE WITH DIFFERENT RATE OF LOADINGS (35)

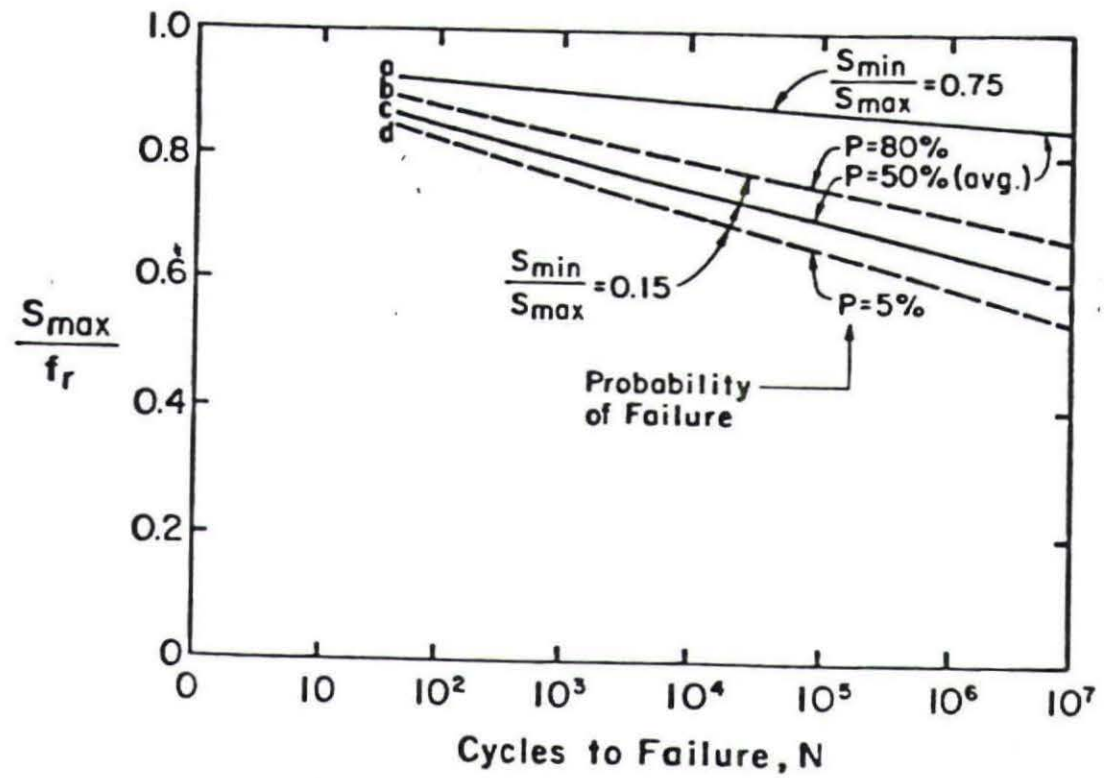


Fig.5 - FATIGUE STRENGTH OF CONCRETE WITH DIFFERENT RANGE OF STRESSES (20)

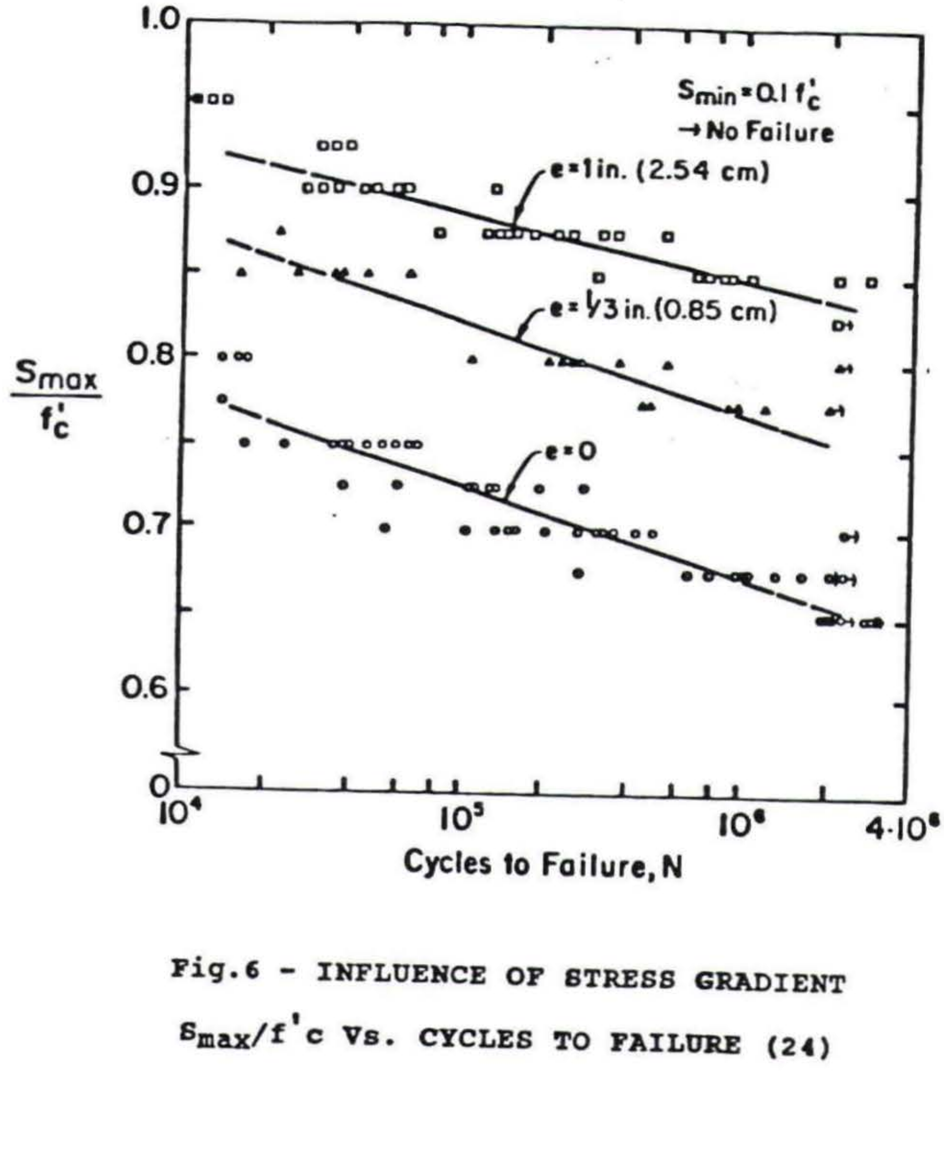


Fig.6 - INFLUENCE OF STRESS GRADIENT

S_{max}/f'_c Vs. CYCLES TO FAILURE (24)

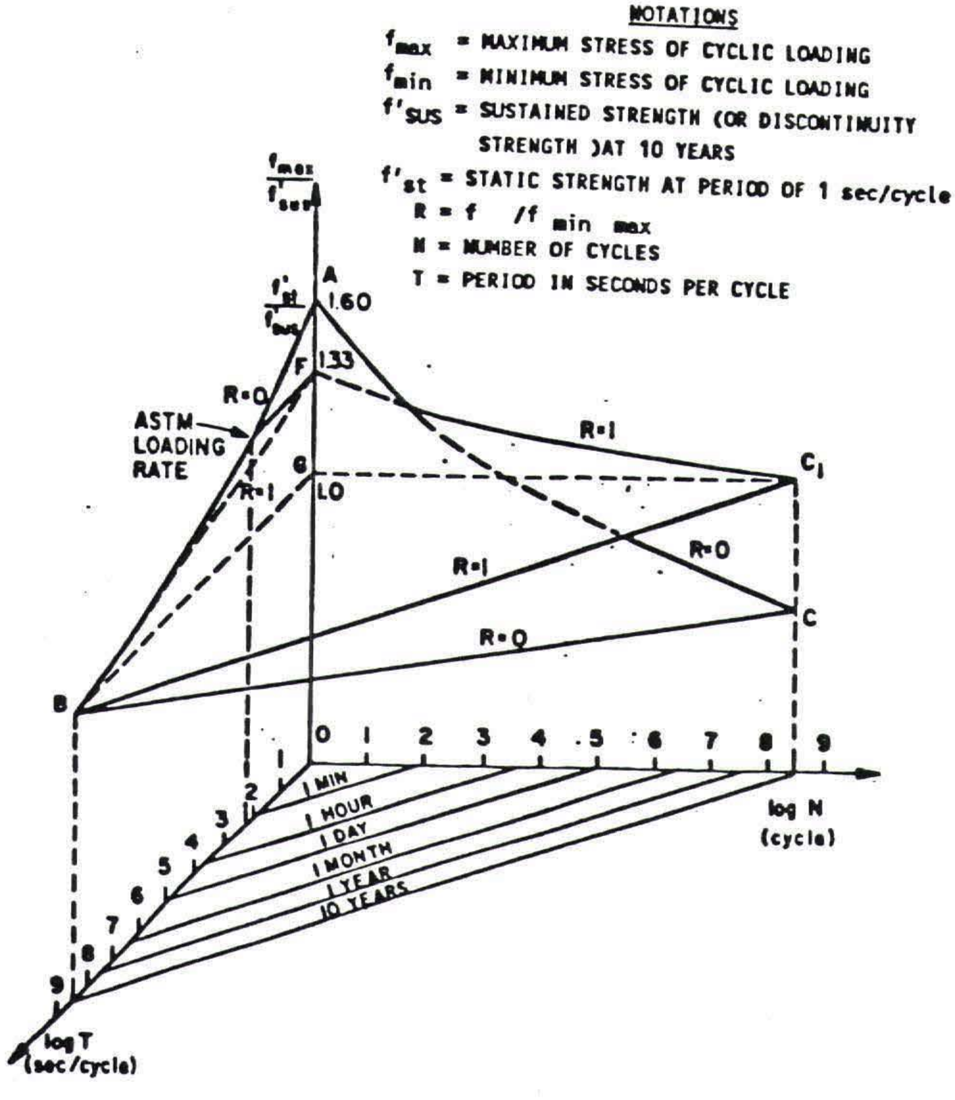


Fig.7 - GRAPHICAL REPRESENTATION OF f-N-T-R RELATION (20).

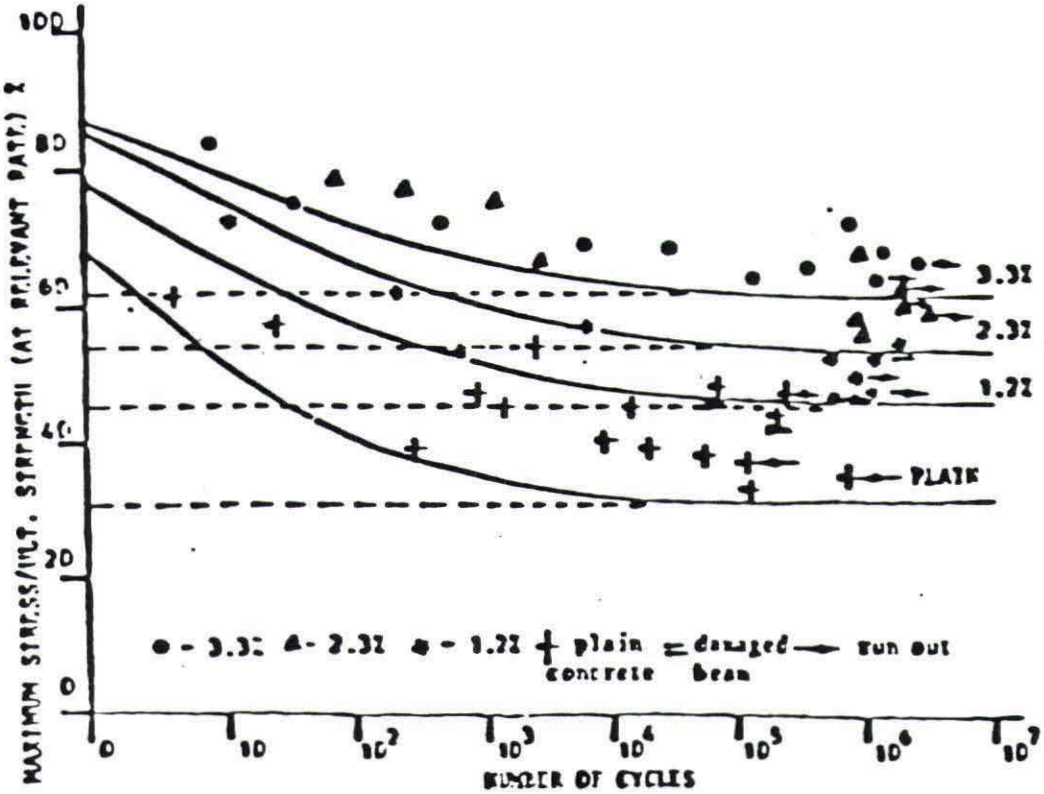


Fig. 8 - FATIGUE STRENGTH Vs. NUMBER OF CYCLES (39).

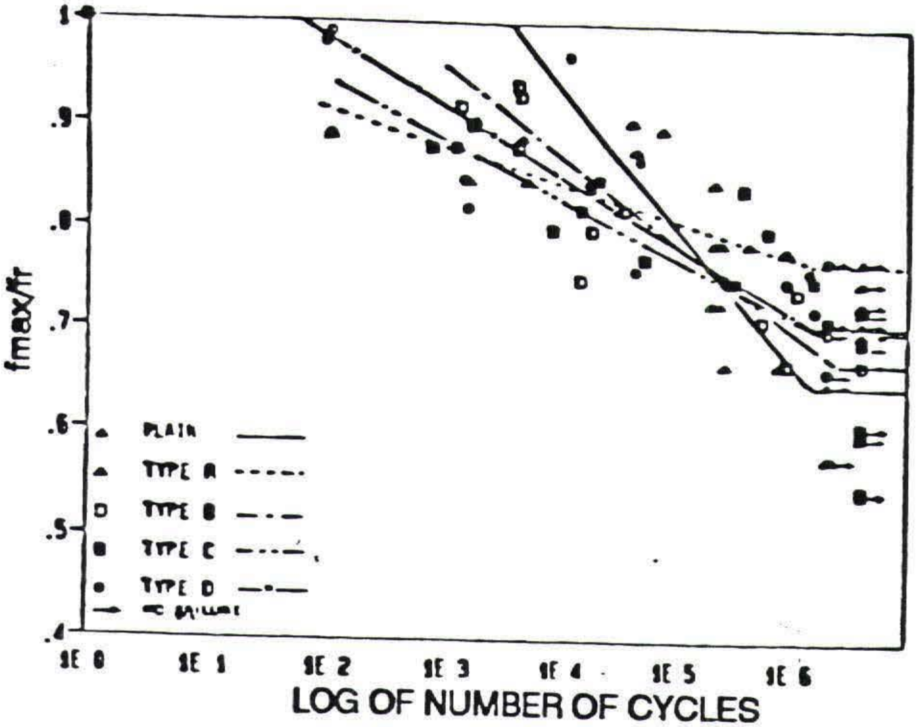


Fig. 9 - RATIO OF FATIGUE STRENGTH TO FLEXURAL STRESS VERSUS

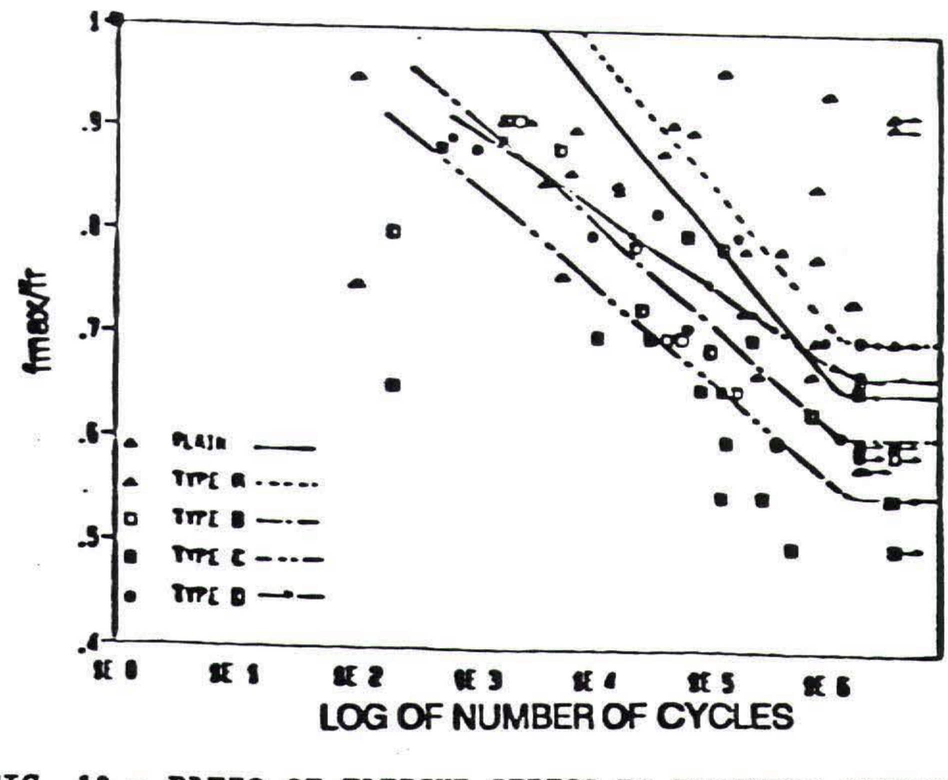


FIG. 10 - RATIO OF FATIGUE STRESS TO FLEXURAL STRESS VERSUS

LOG N FOR 1.0% FIBER BY VOLUME BEAM (16).

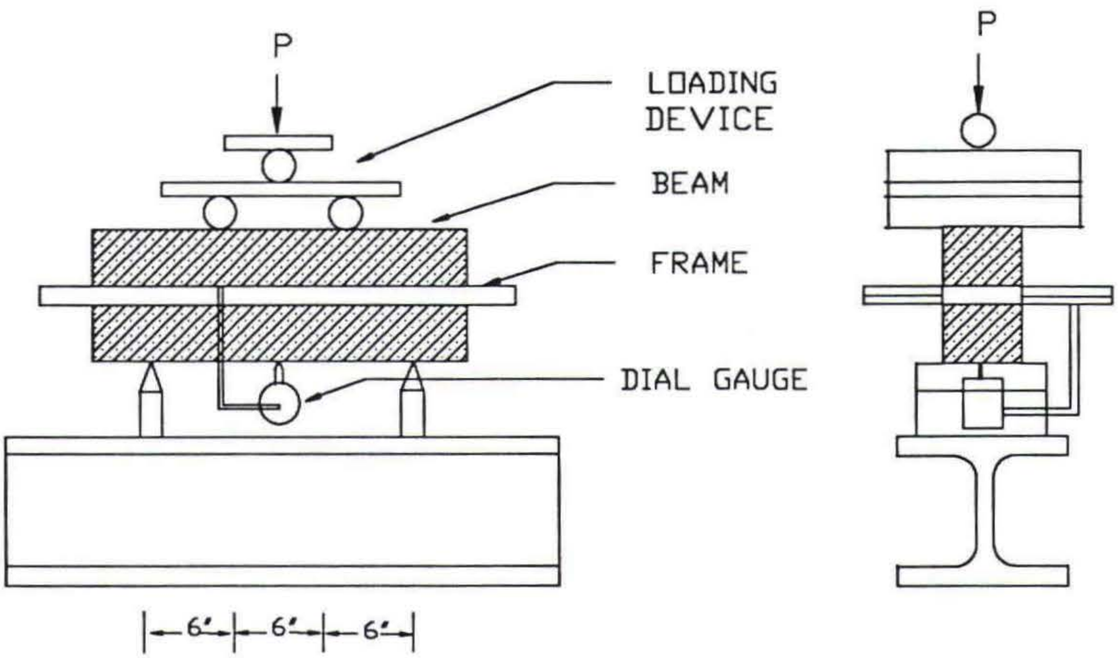
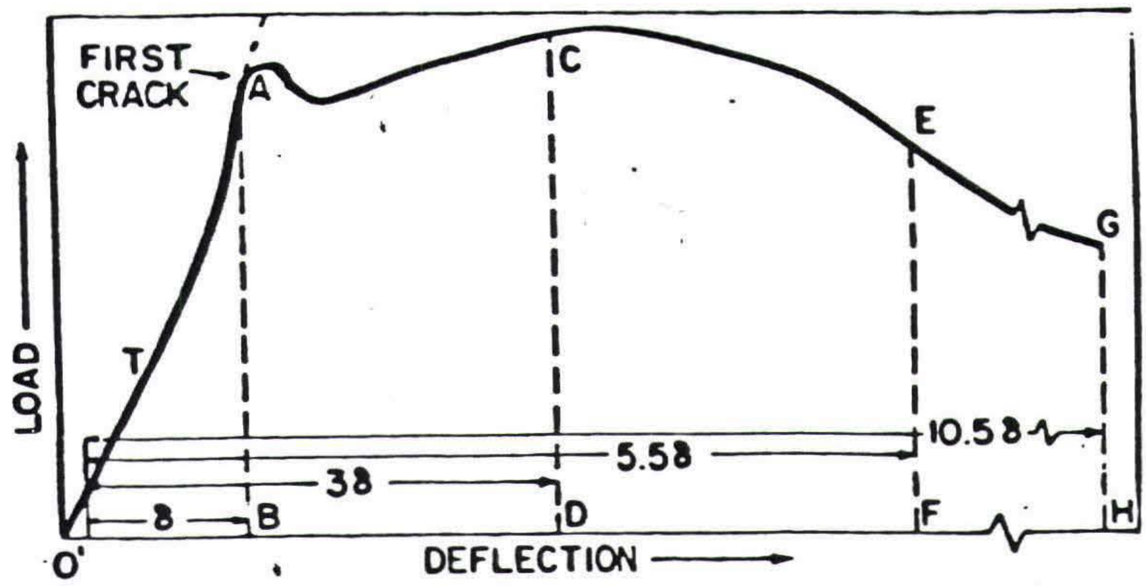


FIGURE 11, TEST SETUP FOR FLEXURE TEST



Concave upwards to first crack

Fig. 12 IMPORTANT CHARACTERISTICS OF LOAD-DEFLECTION CURVE.

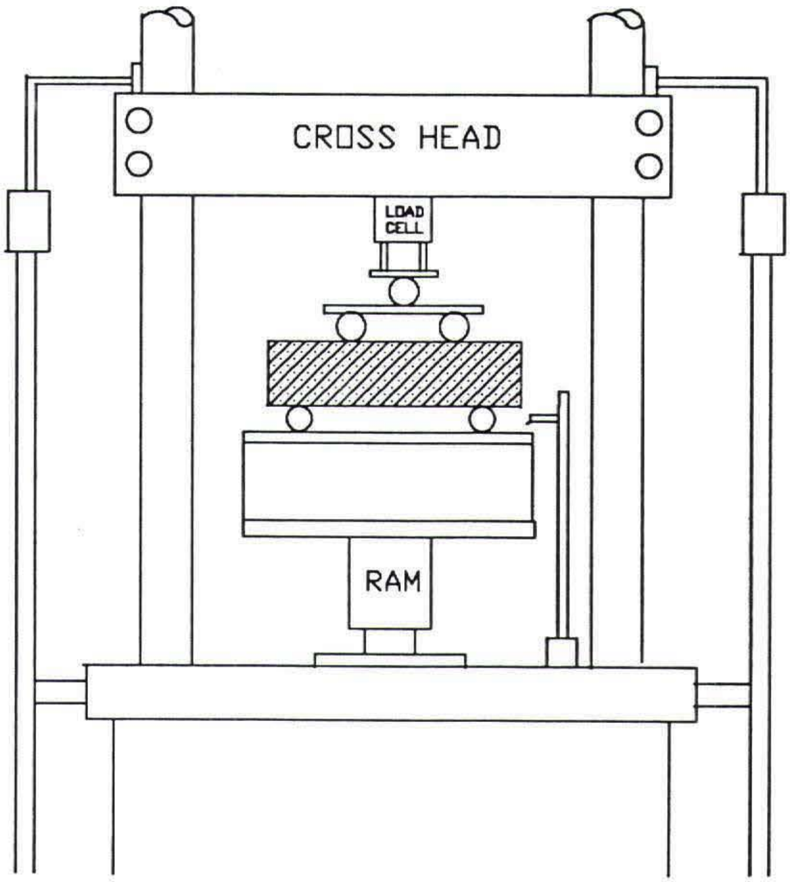


FIGURE 13, FLEXURAL FATIGUE TEST SETUP

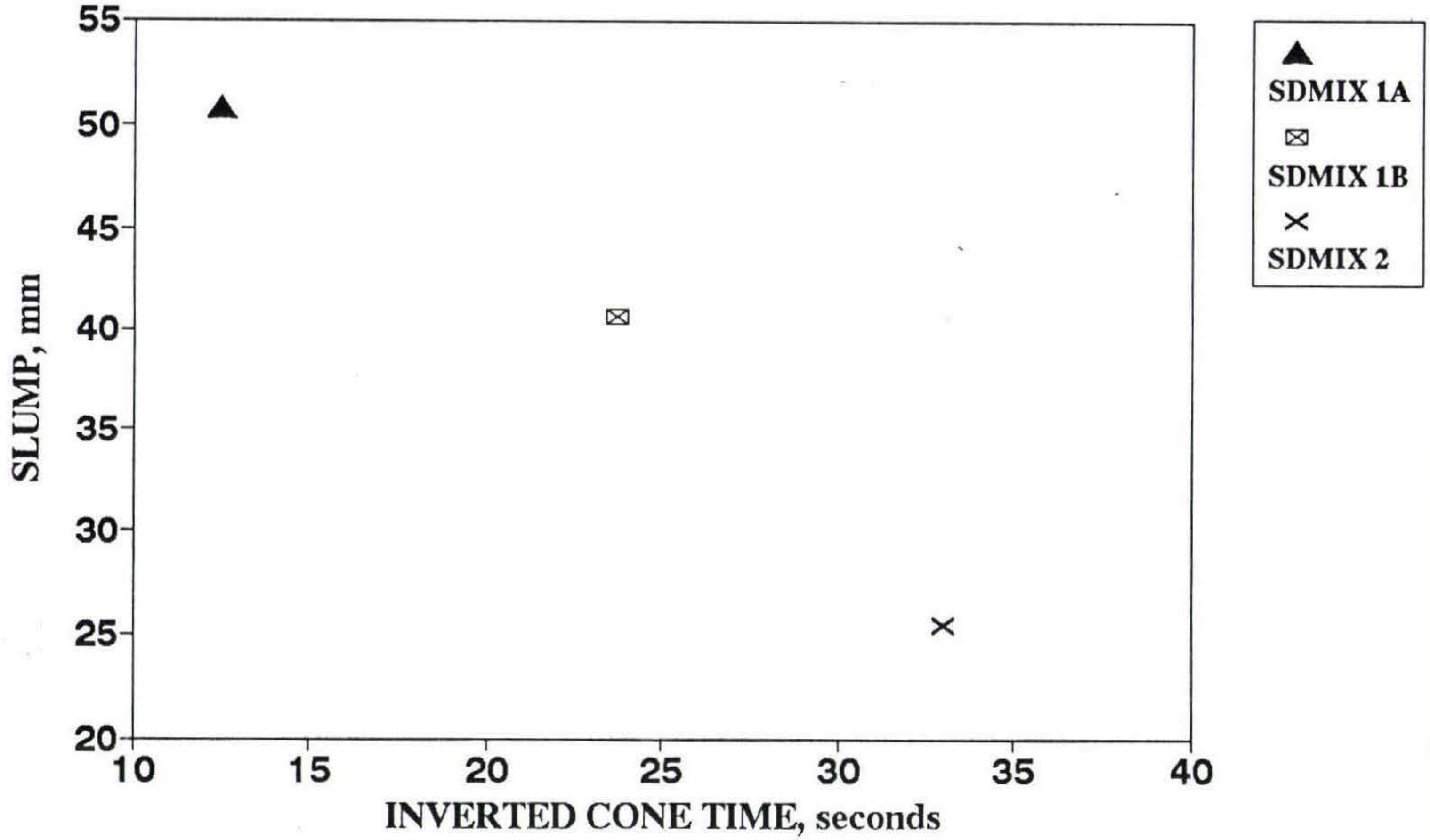


Fig. 14

SLUMP VS INVERTED CONE TIME

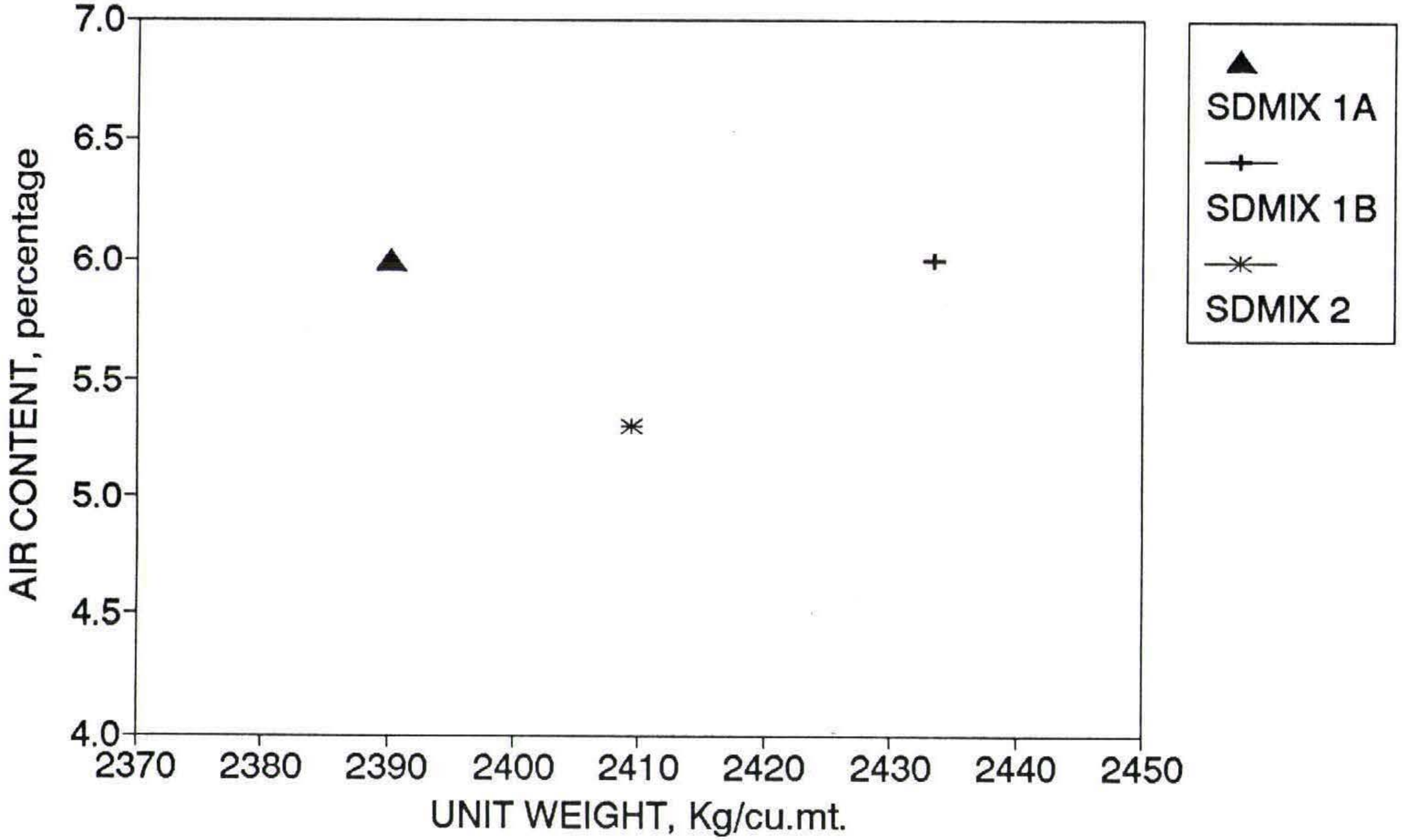


Fig. 15

UNIT WEIGHT Vs. AIR CONTENT

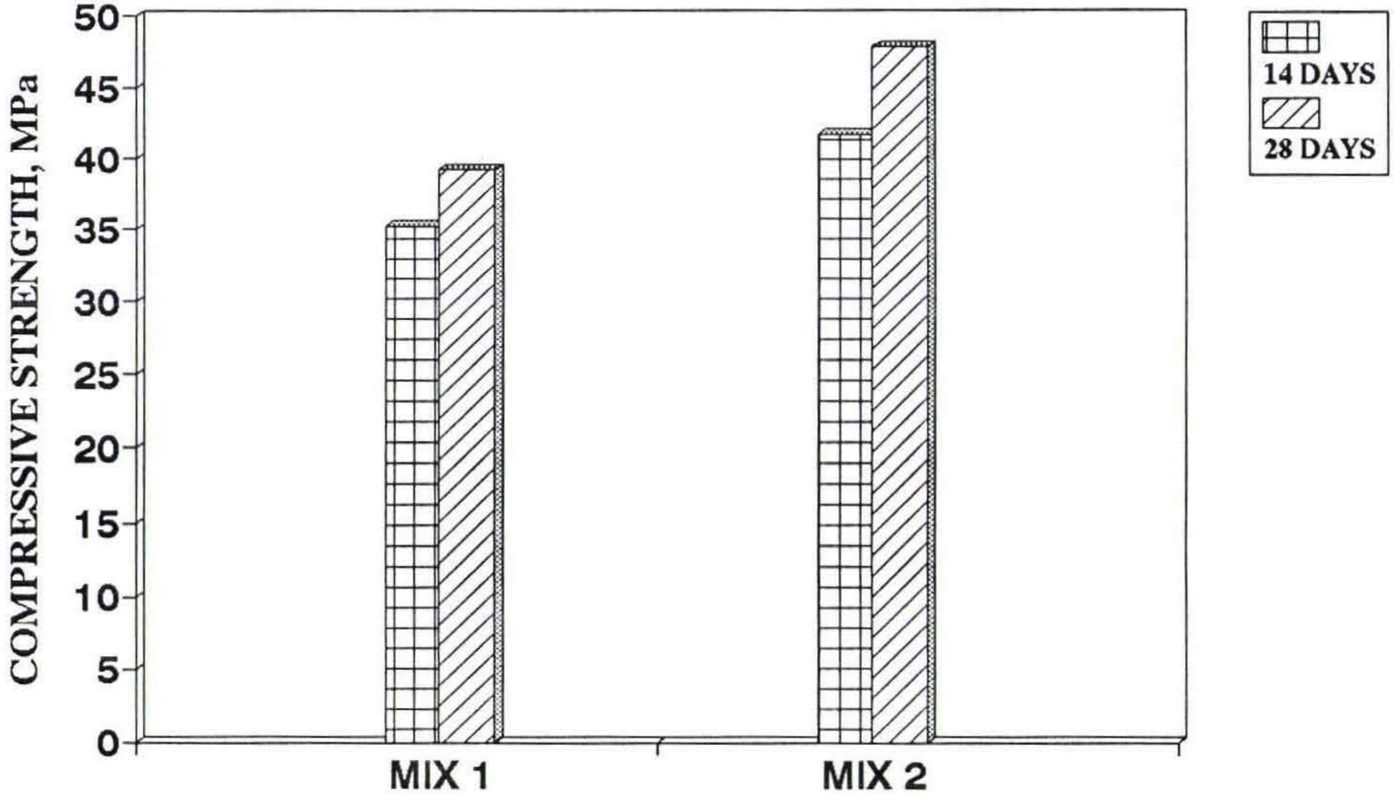


Fig. 16

INCREASE IN COMPRESSIVE STRENGTH FROM 14 TO 28 DAYS

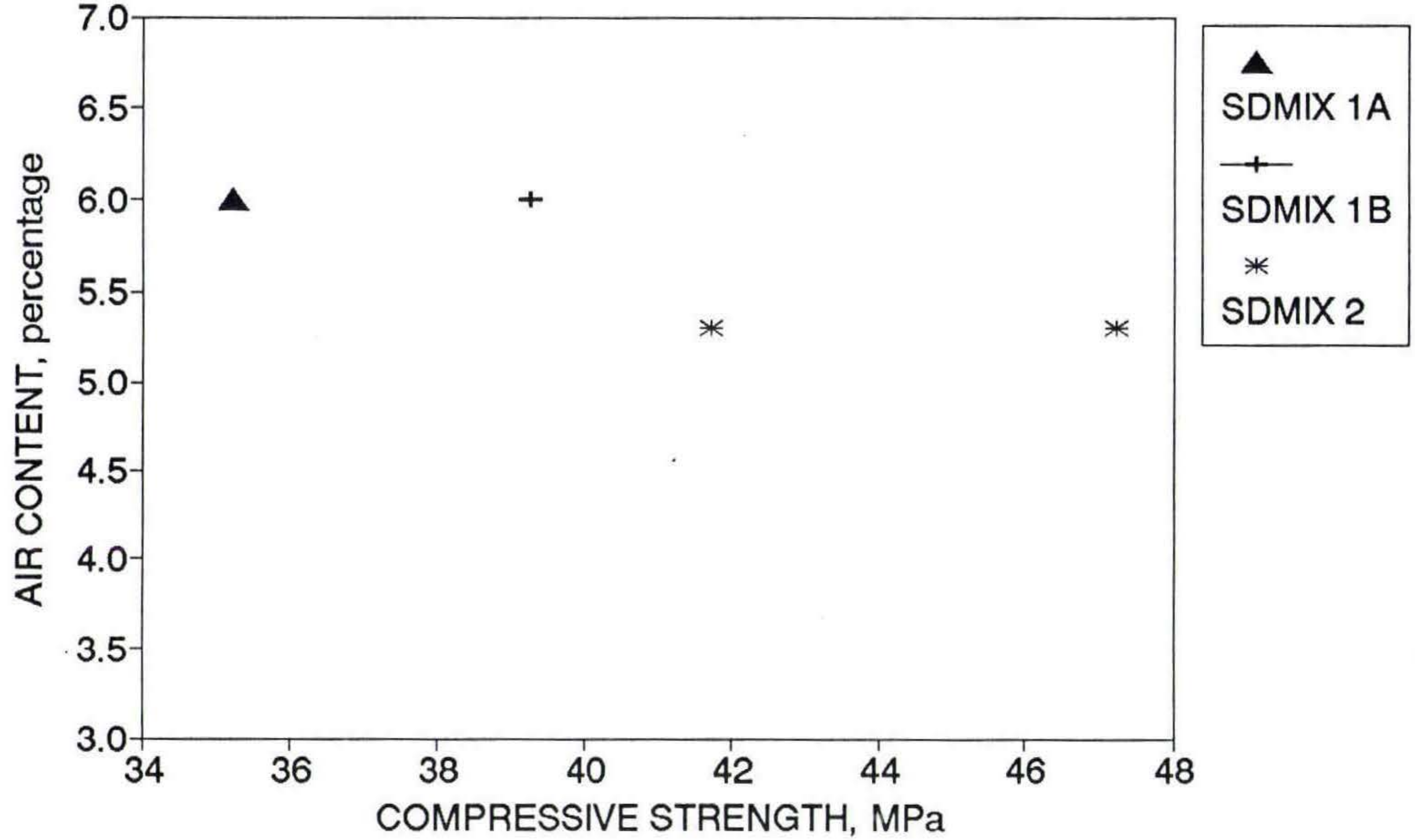


Fig. 17 COMPRESSIVE STRENGTH Vs AIR CONTENT

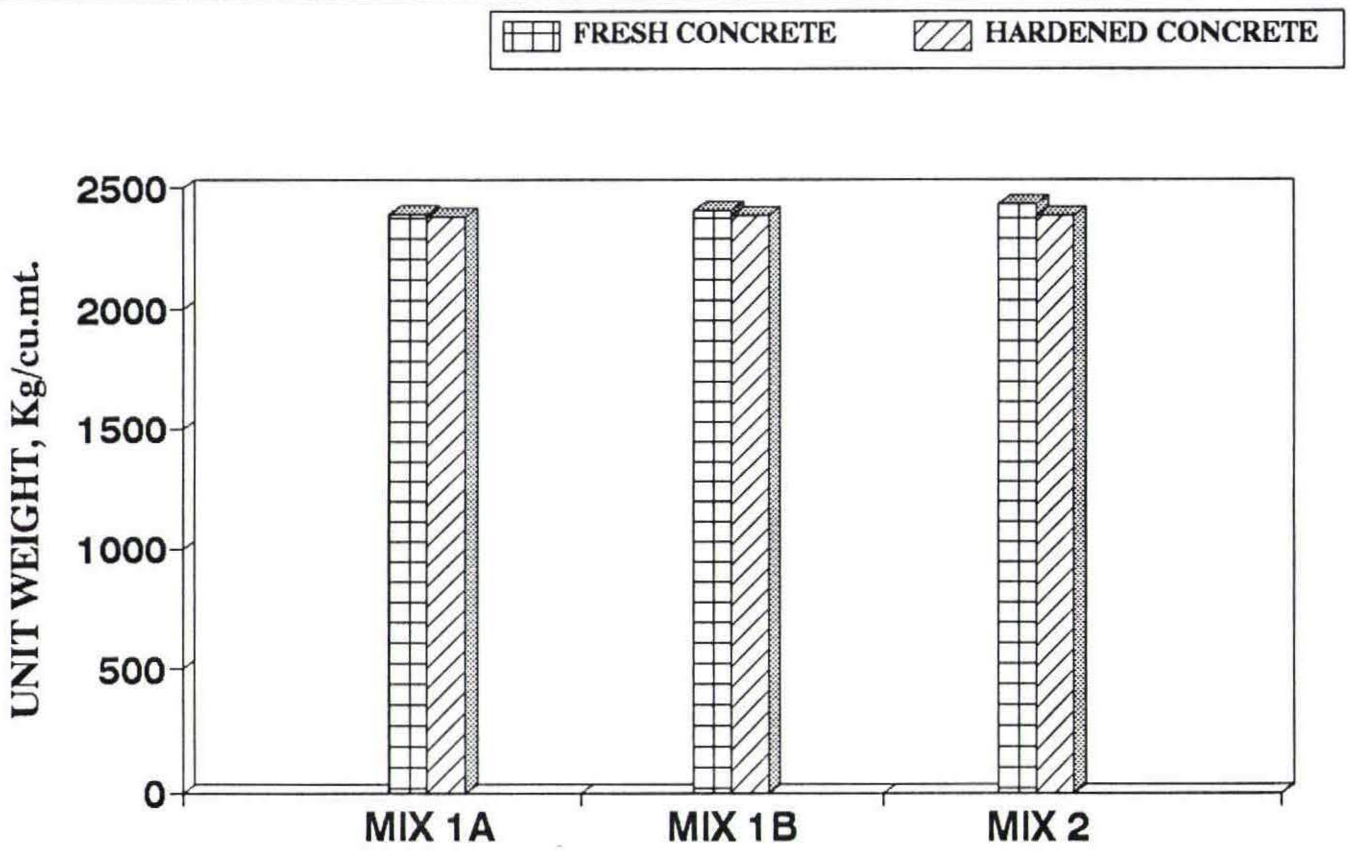


Fig. 18

UNIT WEIGHT OF FRESH CONCRETE Vs HARDENED CONCRETE

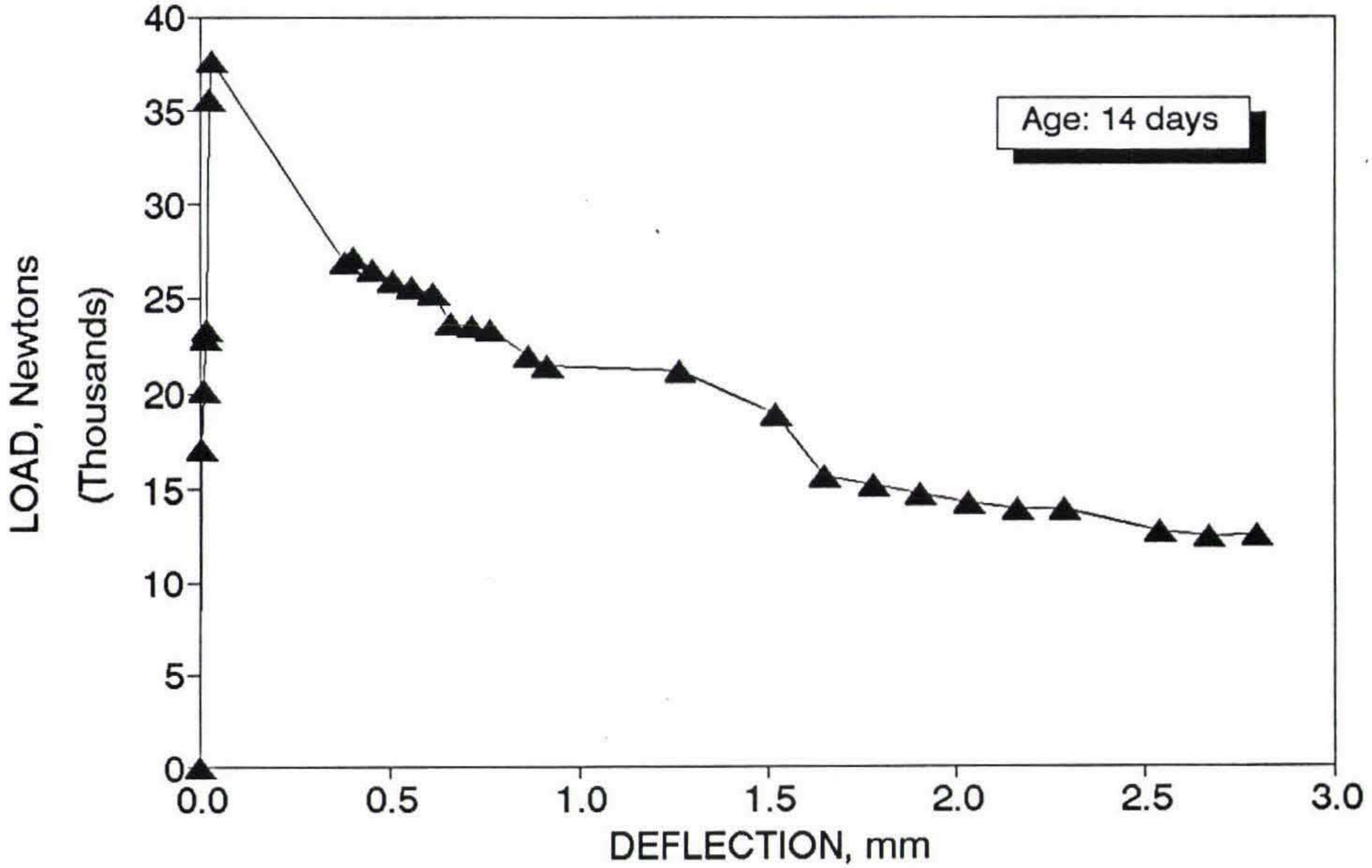


Fig. 19

LOAD DEFLECTION CURVE FOR SDSPB-3

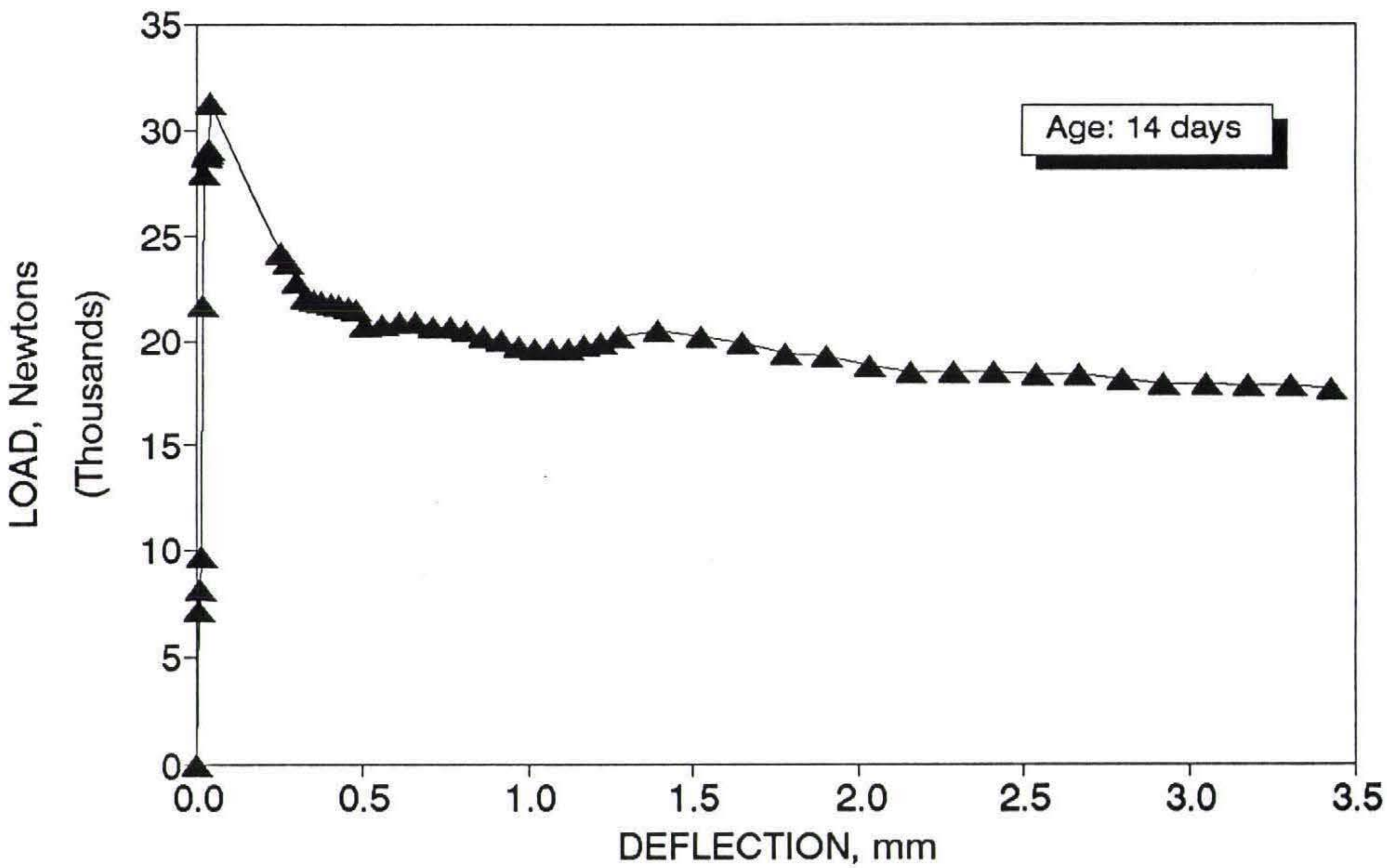


Fig. 20

LOAD DEFLECTION CURVE FOR SDSPB-5

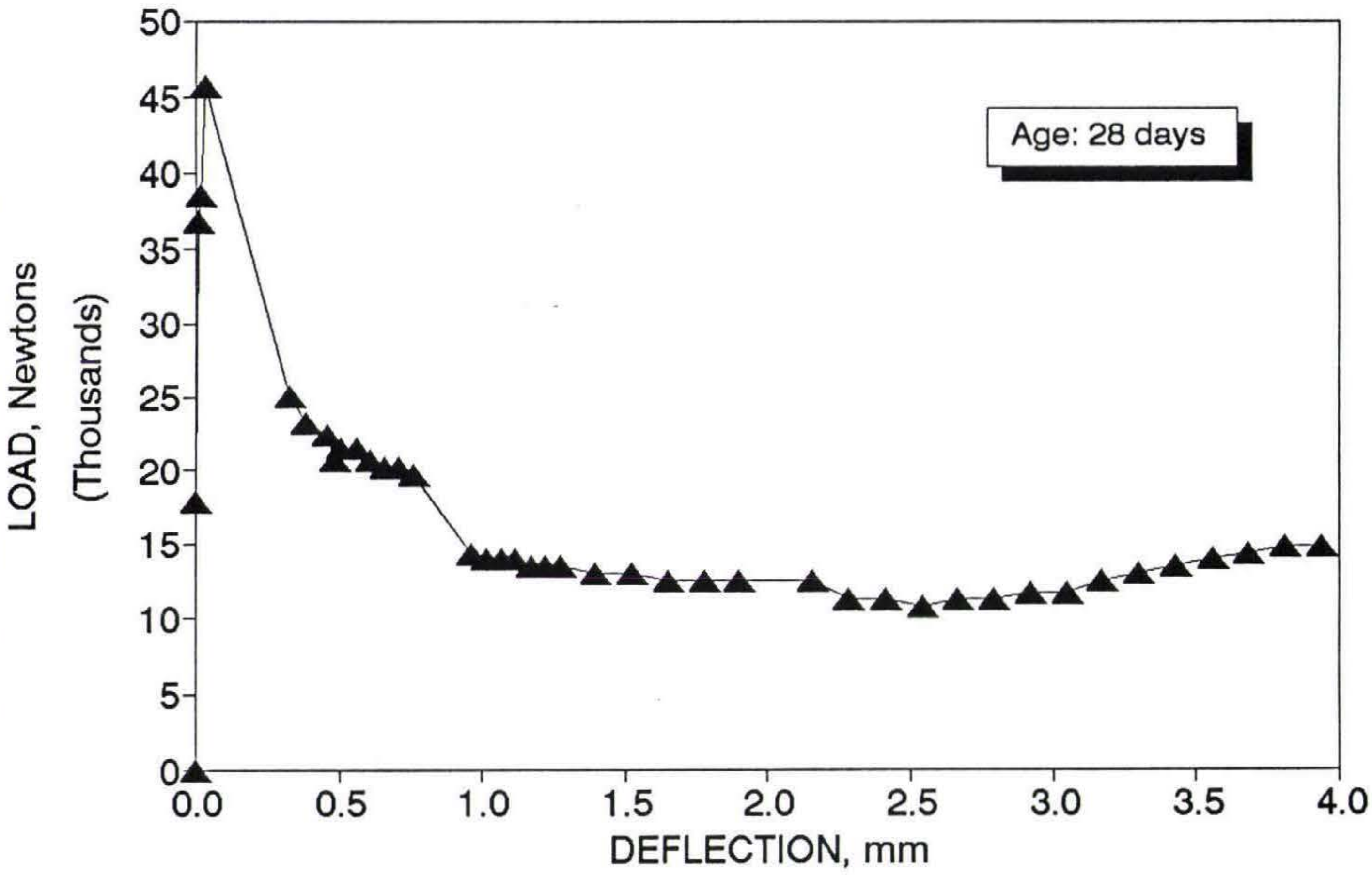


Fig. 21

LOAD DEFLECTION CURVE FOR SDSPB-7

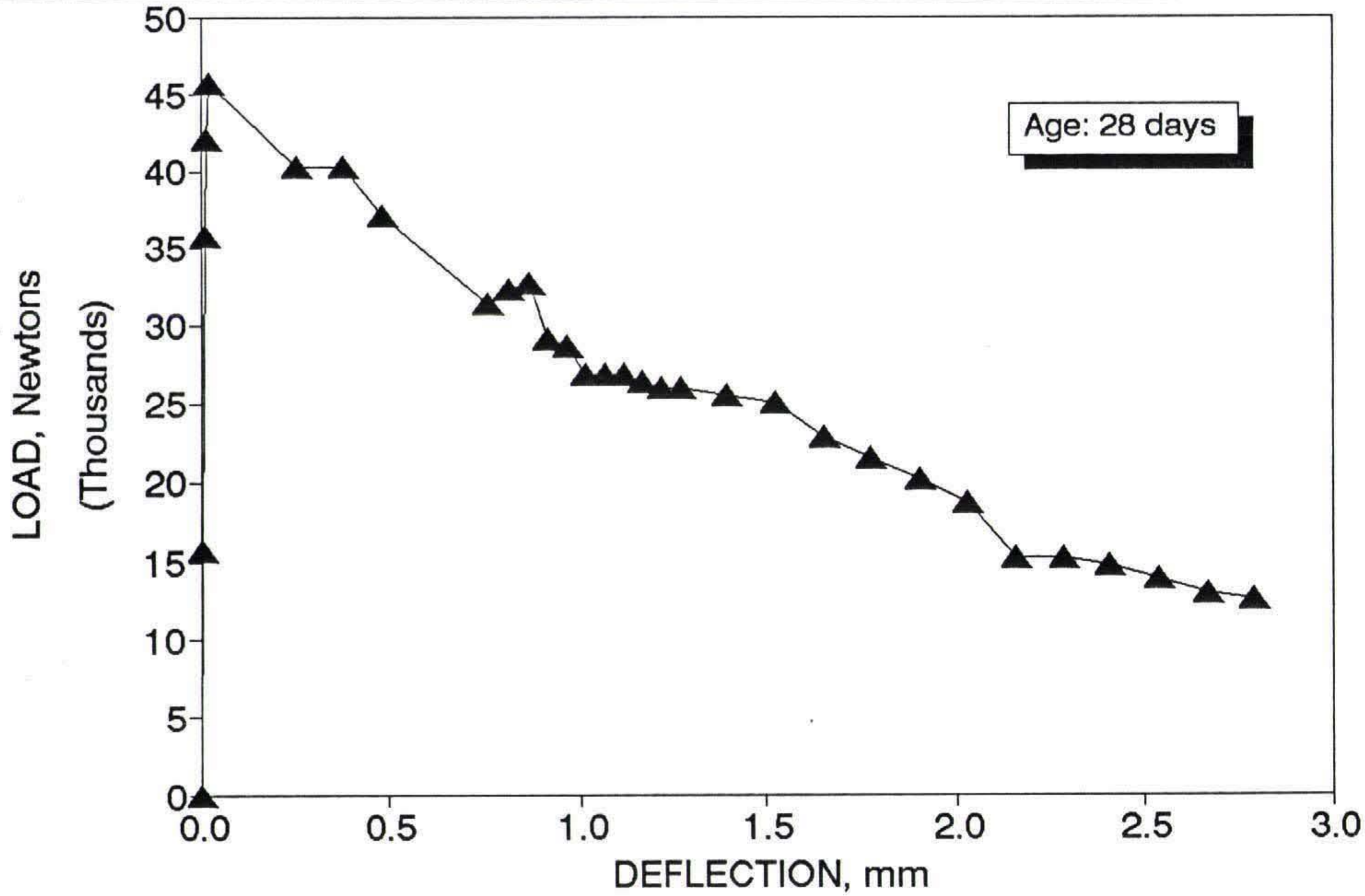


Fig. 22

LOAD DEFLECTION CURVE FOR SDSPB-6

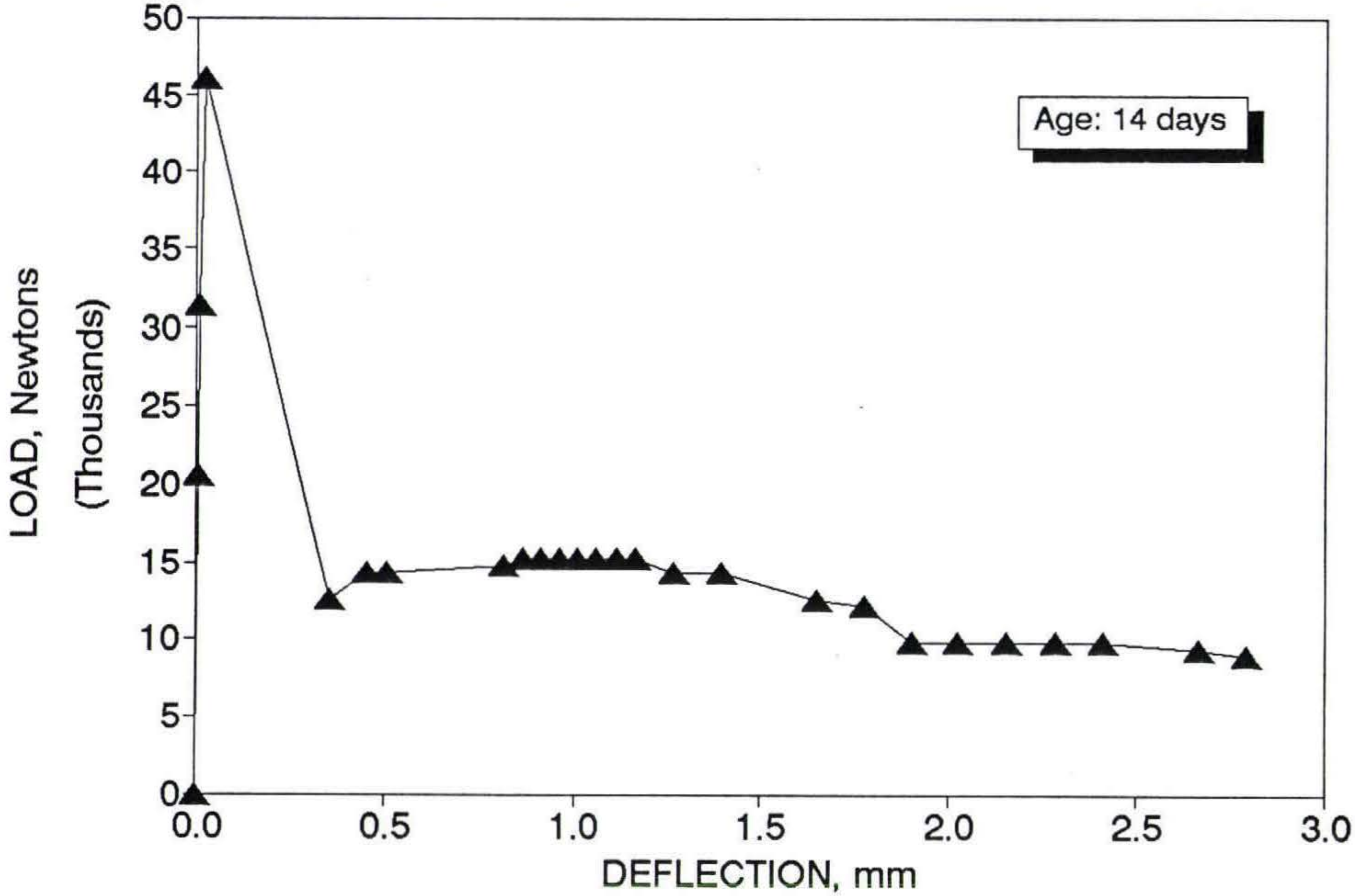


Fig. 23

LOAD DEFLECTION CURVE FOR SDSPB-10

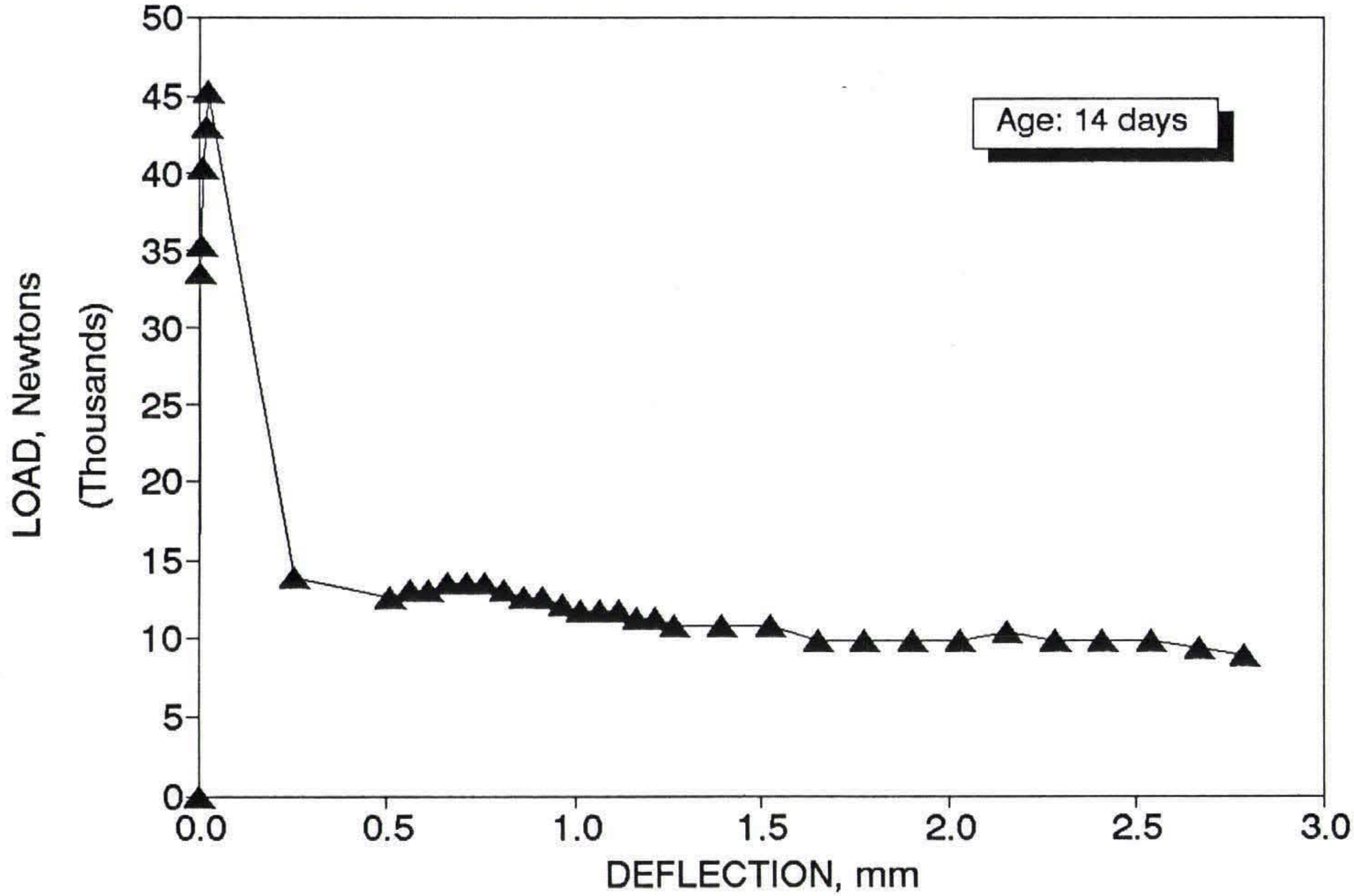


Fig. 24

LOAD DEFLECTION CURVE FOR SDSPB-15

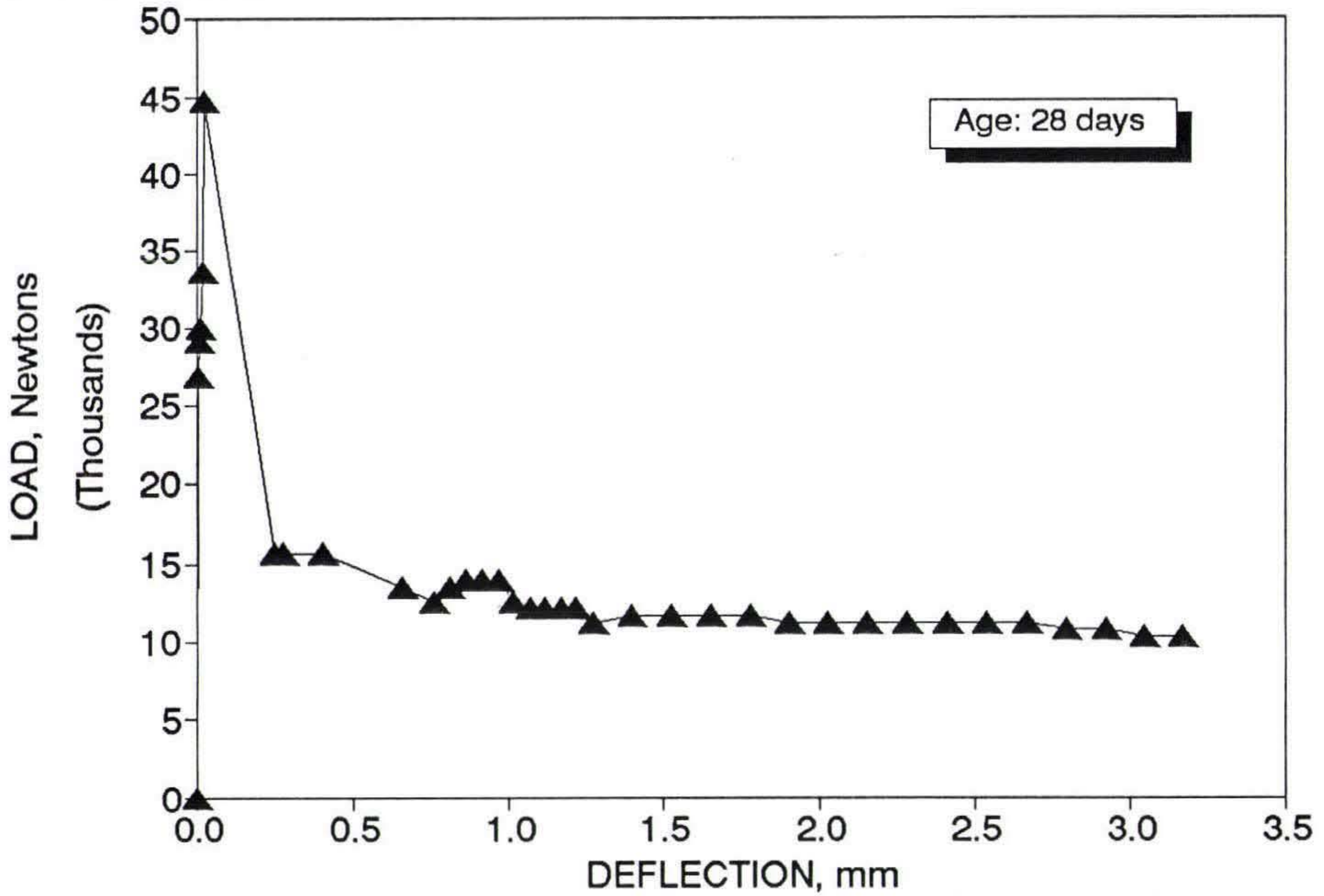


Fig. 25

LOAD DEFLECTION CURVE FOR SDSPB-9

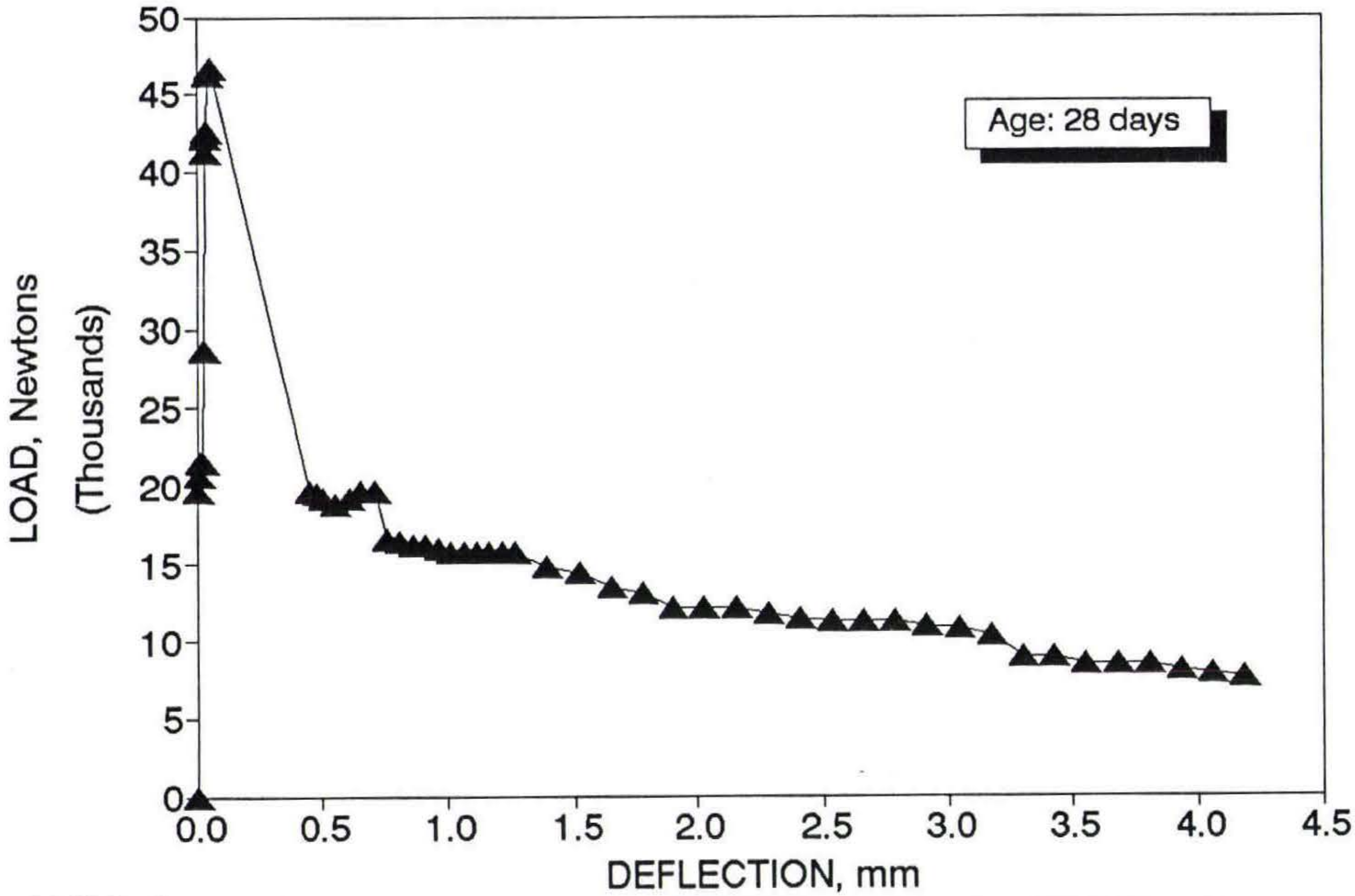


Fig. 26

LOAD DEFLECTION CURVE FOR SDSPB-11

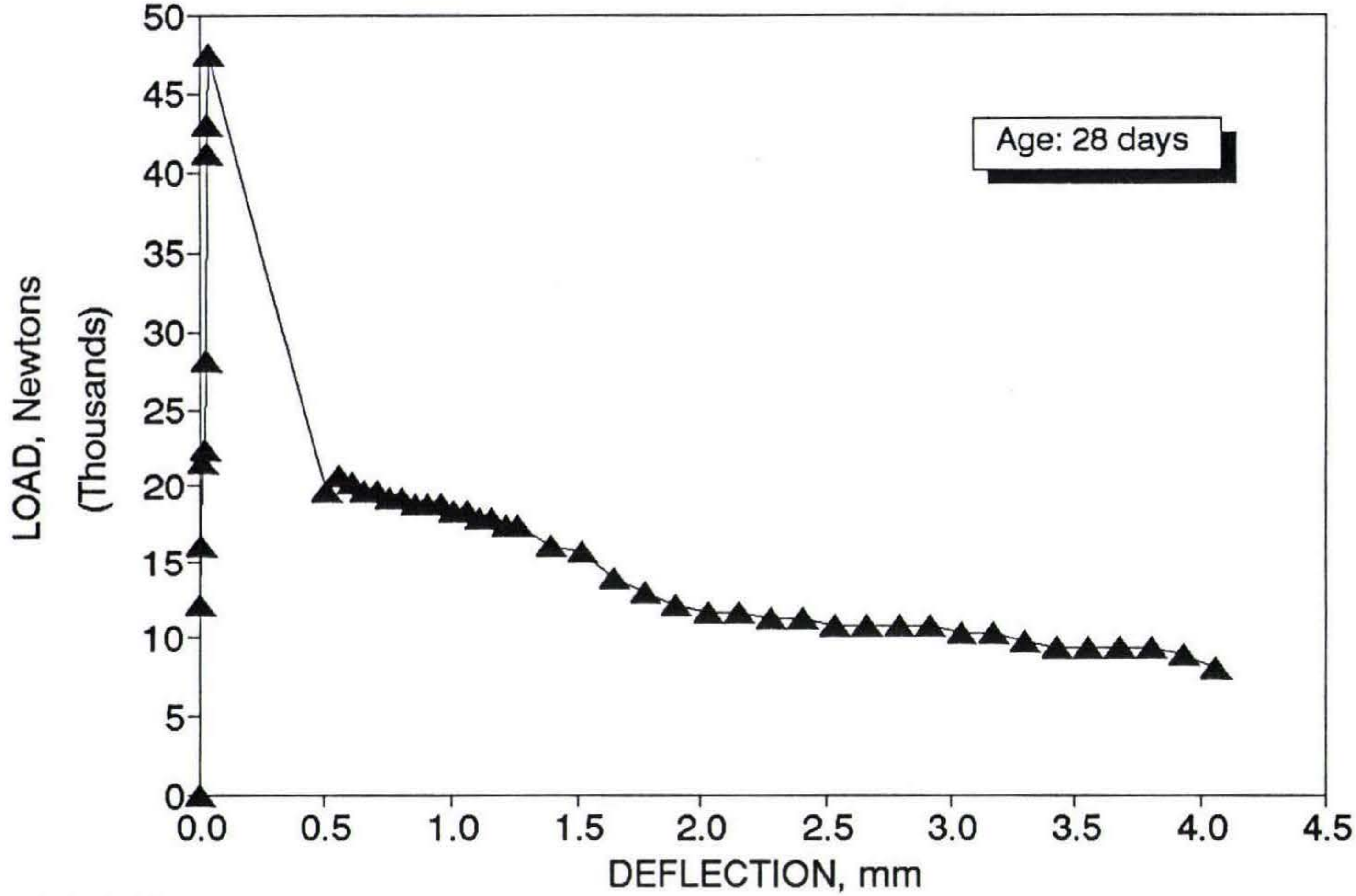


Fig. 27

LOAD DEFLECTION CURVE FOR SDSPB-12

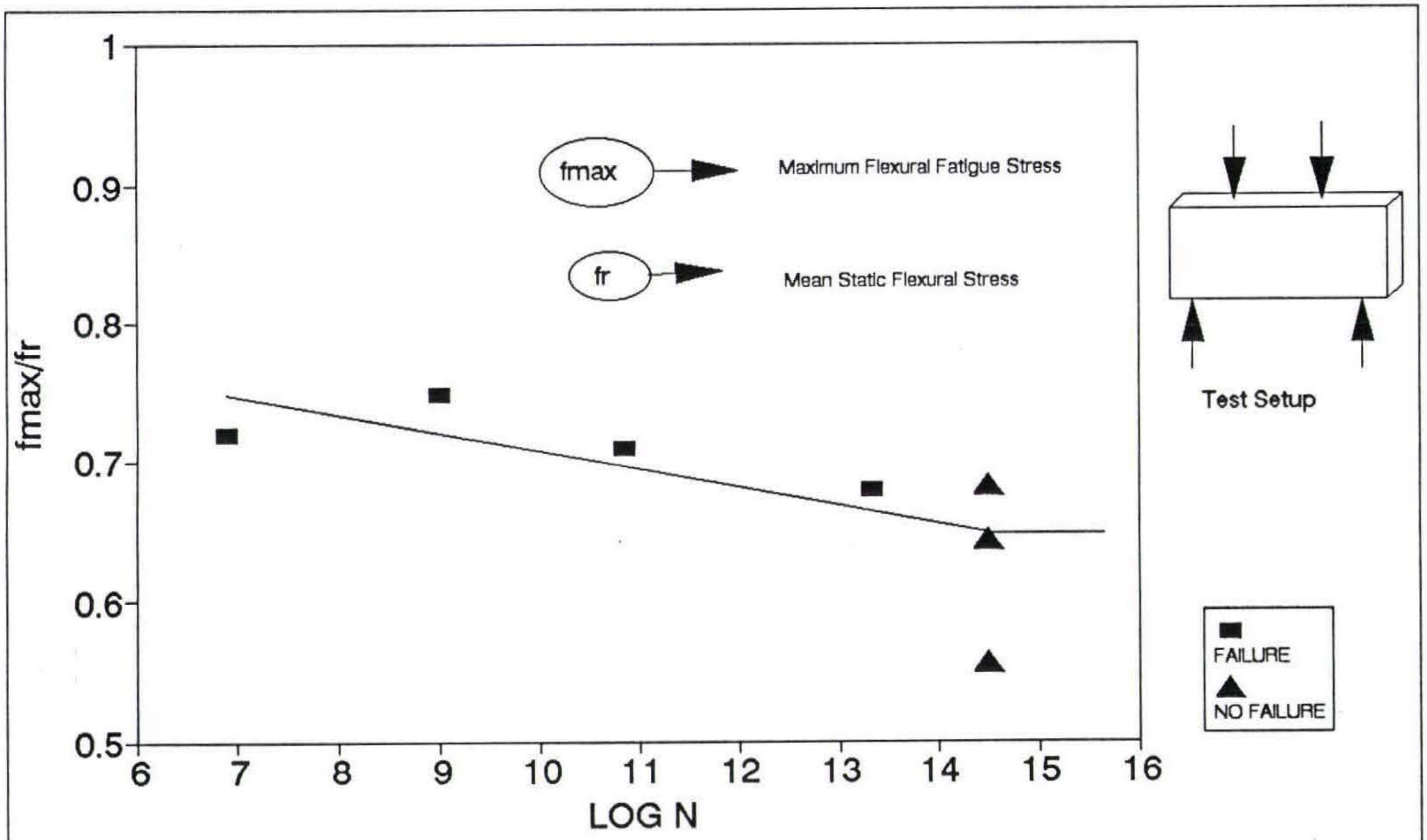


Fig. 28

LOG OF NO. OF CYCLES Vs fmax/fr

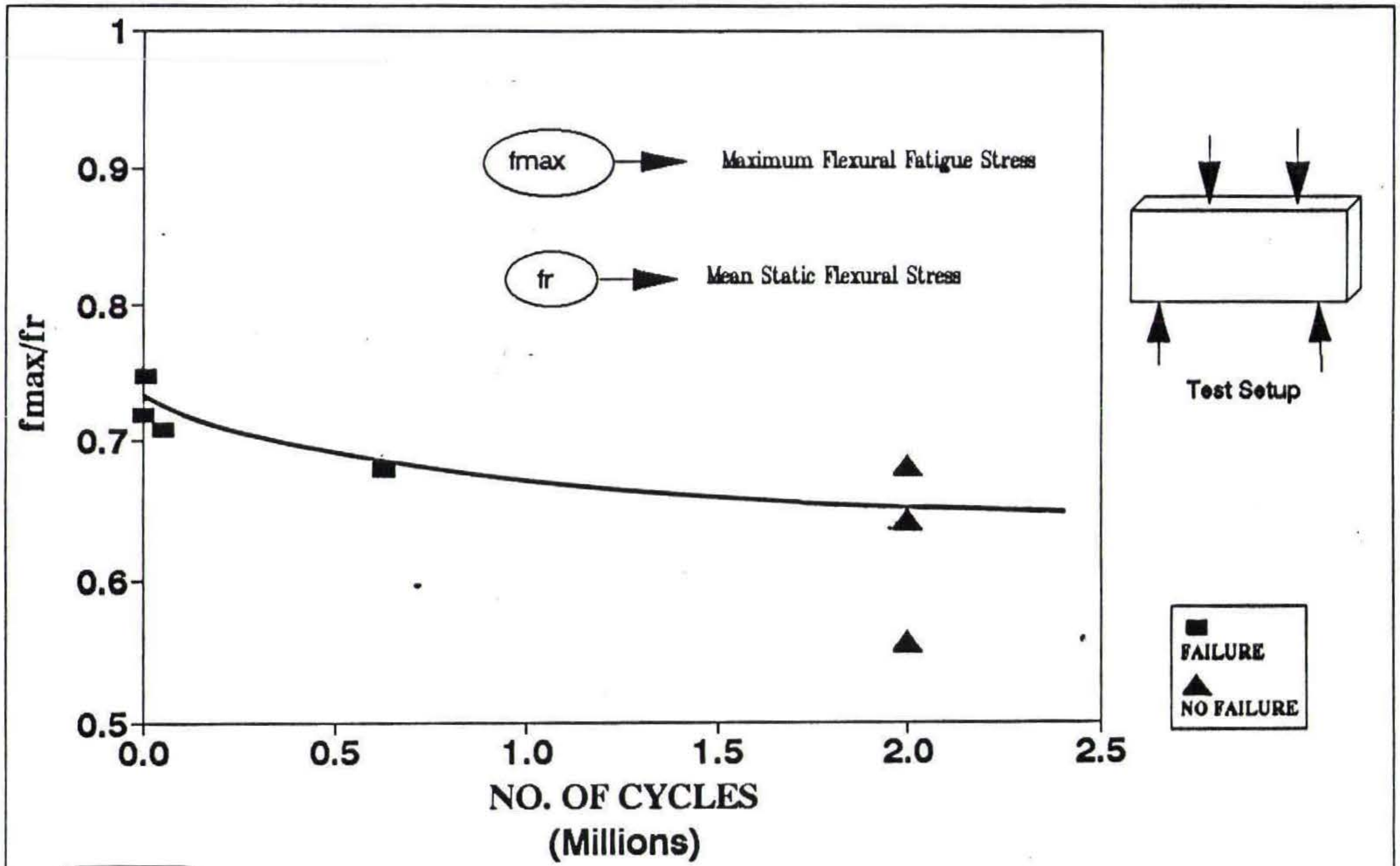


Fig. 29

NO. OF CYCLES Vs fmax/fr

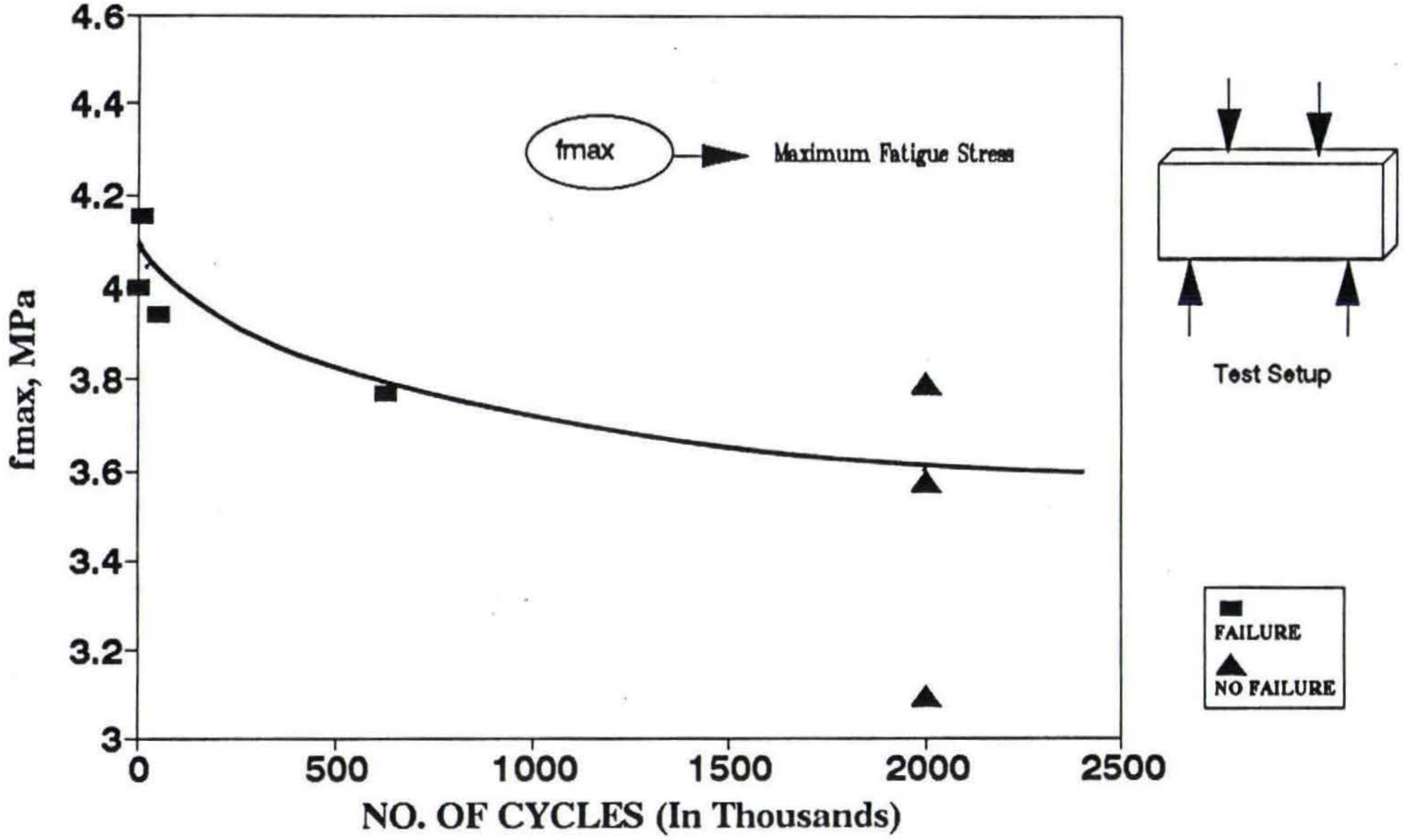


Fig. 30

NO. OF CYCLES Vs fmax

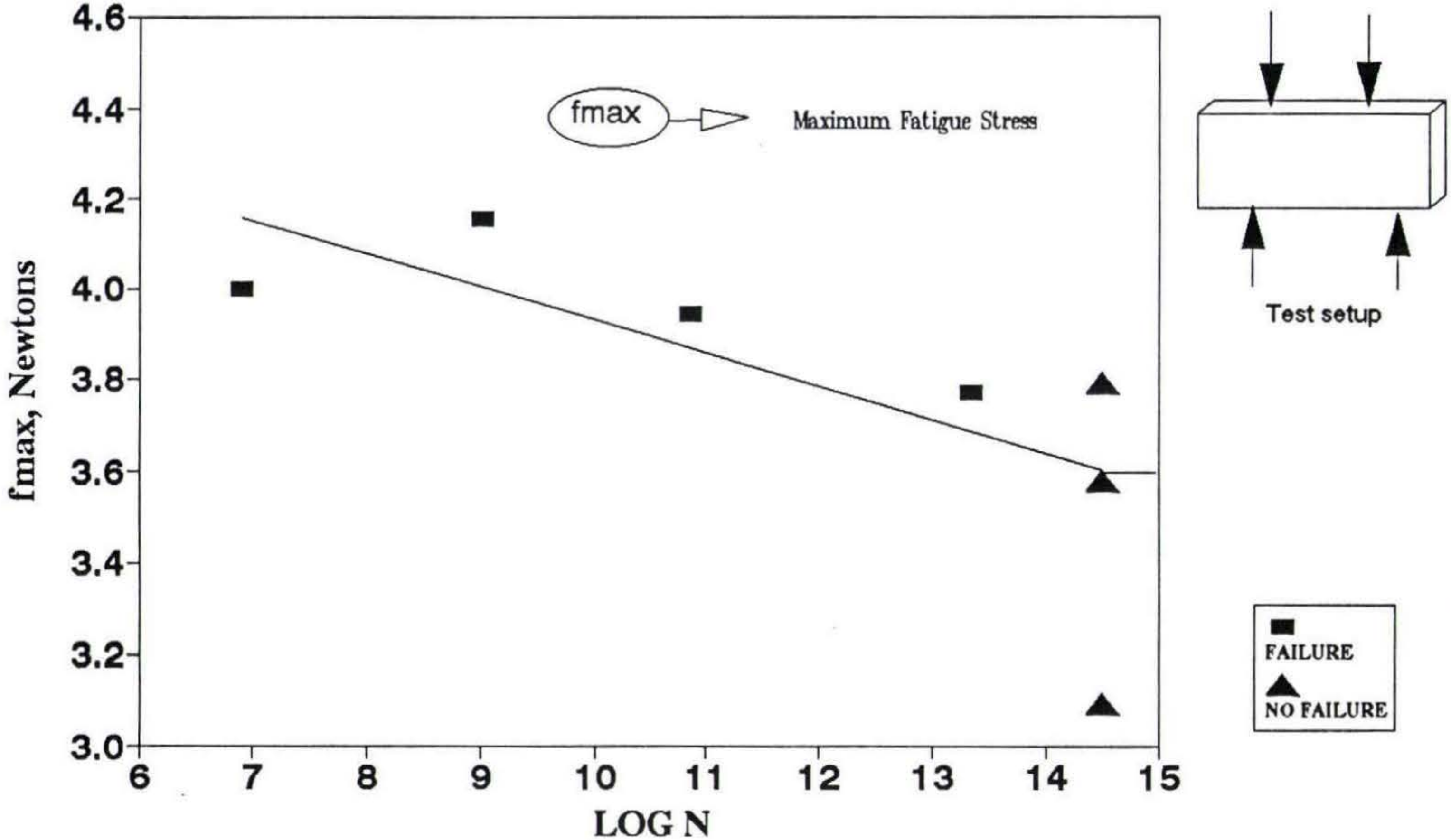


Fig. 31

LOG OF NO. OF CYCLES Vs fmax

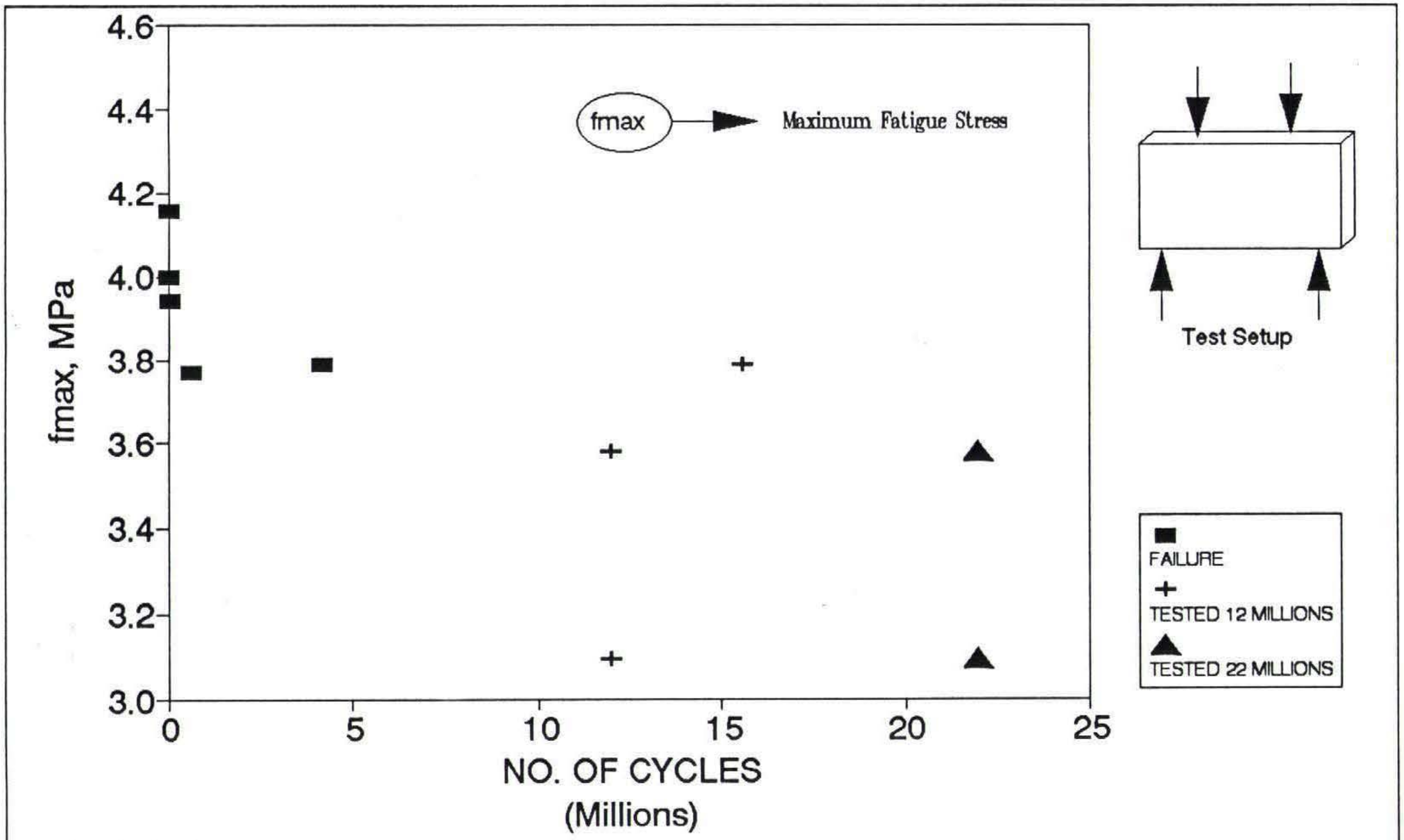


Fig. 32

NO. OF CYCLES Vs fmax

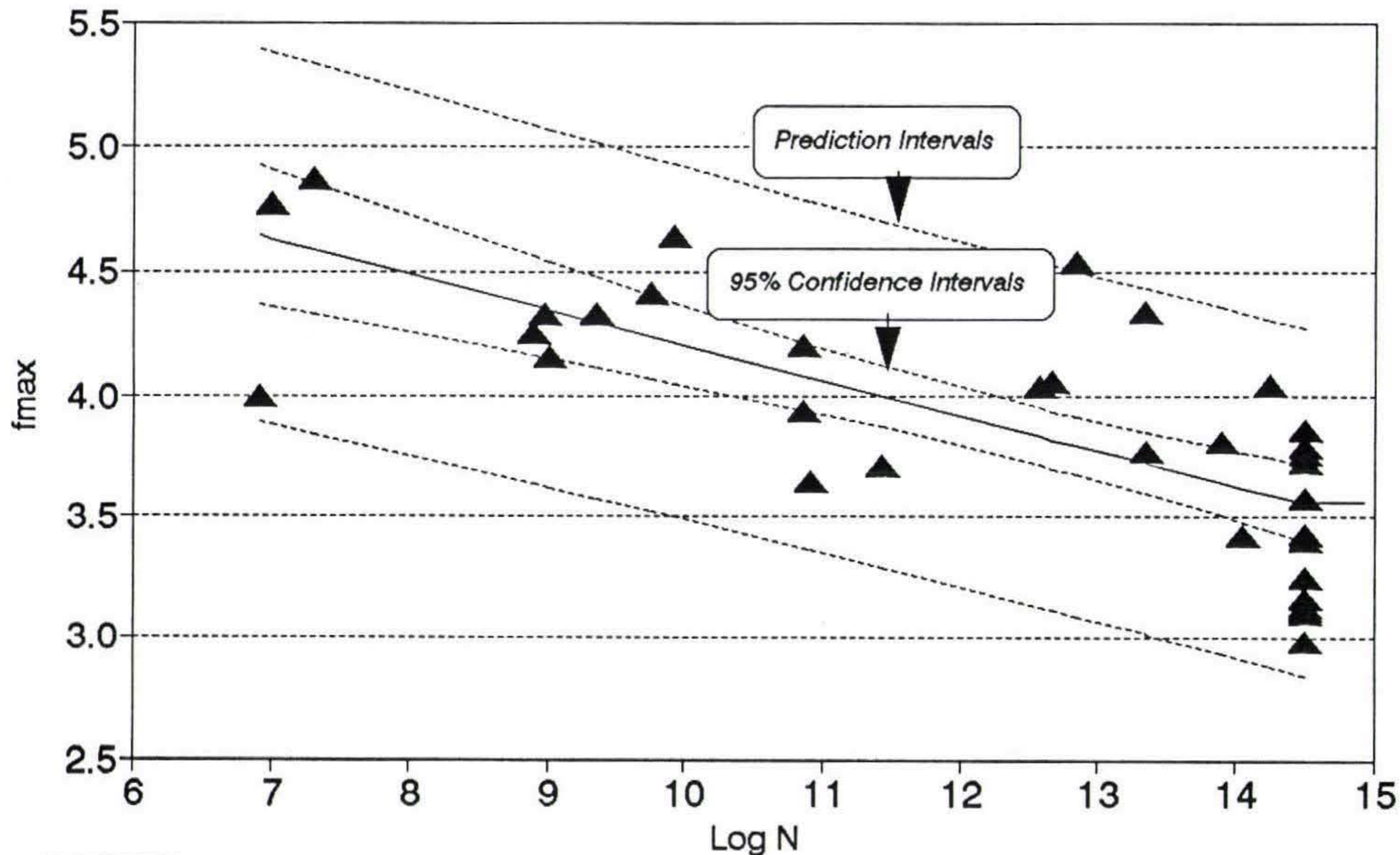
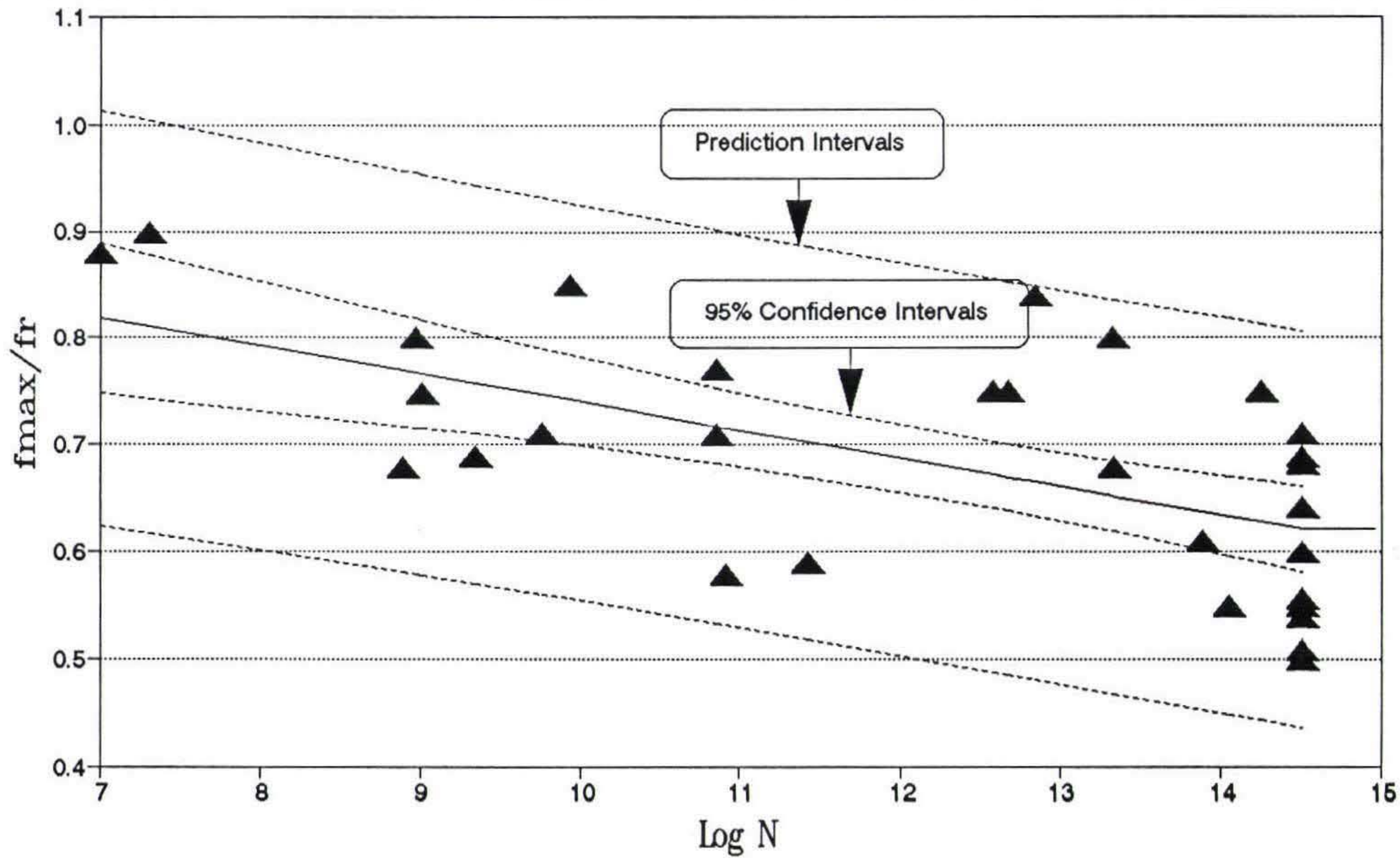


Fig. 33

S-N CURVE



S-N CURVE

Fig. 34

Appendix I

TABLE A. CONCRETE WITH 0.5% CORRUGATED STEEL FIBERS

Fiber properties
Fiber type: Corrugated steel
Aspect ratio: 40 to 65
Tensile Strength: 860 to 1725 MPa
Length: 50.8 mm
Volume: 0.5%

Concrete Properties (MPa)
Ave. com. st. plain:
Ave. com. st.fiber: 48.31
Ave. fle. st. plain:
Ave. fle. st. fiber: 6.25

SP. #	Maximum Stress (MPa)	f_{fmax}/f_r	Cycles to failure
RC5I-1 @	0	0.00	0
RC5II-1 $	0	0.00	0
RC5III-1 &	0	0.00	0
RC5I-2	5.571	0.89	100
RC5II-2	5.536	0.89	100
RC5III-2	5.592	0.89	300
RC5I-4	4.254	0.68	7300
RC5II-4	4.413	0.71	17300
RC5III-4	4.330	0.69	11500
RC5I-5	3.709	0.59	92300
RC5II-5	3.813	0.61	1078700
RC5III-5	3.647	0.58	55200
RC5I-3	3.406	0.54	2000000 + α
RC5II-3	3.419	0.55	1267000
RC5III-3	3.427	0.55	2000000 + α
RC5I-6	3.165	0.51	2000000 + α
RC5II-6	3.117	0.50	2000000 + α
RC5III-6	3.130	0.50	2000000 + α

@ _ Tested for static load = 38848 Newtons;
Flexural stress = 4.929 MPa;
$ - Maximum static load = 54332 Newtons;
Flexural stress = 6.770 MPa;
& - Maximum static load = 45911 Newtons;
Flexural stress = 5.729 MPa;
+ - No failure.
α - After a lapse of time each beam was further tested up to 4 million cycles and did not fail.
f_r - Average flexural stress ($ and &) = 6.254 MPa;
f_{fmax} - Fatigue flexural maximum stress.

TABLE B. CONCRETE WITH 0.5% CORRUGATED STEEL FIBERS

Fiber properties		**Concrete Properties (MPa)**	
Fiber type:	Corrugated steel	Ave. com. st. plain:	
Aspect ratio:	40 to 65	Ave. com. st. fiber:	47.09
Tensile Strength:	860 to 1725 MPa	Ave. fle. st. plain:	
Length:	50.8 mm.	Ave. fle. st. fiber:	5.433
Volume:	0.5%		

SP. #	Maximum Stress (MPa)	f_{fmax}/f_r	Cycles to failure
SC6-III1 θ	0	0.00	0
SC6-I1 β	0	0.00	0
SC6-II5	4.875	0.90	1500
SC6-I2	4.771	0.88	1100
SC6-I5	4.799	0.88	700
SC6-I4	4.330	0.80	7900
SC6-III4	4.206	0.77	52000
SC6-II4	4.399	0.80	617700
SC6-III6	4.047	0.75	1542400
SC6-III3	4.040	0.75	289800
SC6-III5	4.062	0.75	319500
SC6-I6	4.537	0.84	379900
SC6-II6	4.640	0.85	20500
SC6-II2	3.730	0.69	2000000 + α
SC6-I3	3.862	0.71	2000000 + α
SC6-II3	3.765	0.69	2000000 + *
SC6-III2	3.254	0.60	2000000 + α
SC6-II1	2.985	0.55	2000000 + α

θ - Tested for max. Static load = 43261 Newtons;
Flexural stress = 5.454 MPa;
β - Max. Static Load = 42918 Newtons;
Flexural Stress = 5.413 MPa;
+ - No failure
* - Tested for flexure after fatigue.
α - Later tested for Flexural after Fatigue
f_{fmax} - Fatigue maximum stress
f_r - Modulus of Rupture

TABLE C. CONCRETE WITH 0.5% CORRUGATED STEEL FIBERS

Fiber properties
Fiber type: Corrugated steel
Aspect ratio: 40 to 65
Tensile Strength: 860 to 1725 MPa
Length: 50.8 mm
Volume: 0.5%

Concrete Properties (MPa)
Ave. com. st. plain:
Ave. com. st. fiber: 40.5
Ave. fle. st. plain:
Ave. fle. st. fiber: 5.56

SP. #	Maximum Stress (psi)	f_{fmax}/f_r	Cycles to failure
SDSPB-4	4.158	0.748	8200
SDSPB-17	3.999	0.719	1000
SDSPB-5	3.943	0.709	52200
SDSPB-1	3.792	0.682	2000900 +
SDSPB-18	3.799	0.682	2000900 +
SDSPB-14	3.772	0.679	625000
SDSPB-13	3.579	0.643	2000000 +
SDSPB-16	3.096	0.557	2000890 +

+ Beams crossed 2 million cycles and later they were tested for an additional 10 million cycles.

APPENDIX II

The following tables present SI (Systems International) units and the conversion of English units to SI units (ASTM E380-89).

SI base units

Quantity	Name	Symbol
length	meter	m
mass	kilogram	kg
time	second	s
electric current	ampere	A
temperature	kelvin	k
amount of substance	mole	mol
luminous intensity	candela	cd

Conversion Factors to SI Units

English	SI	SI symbol	To convert from English to SI Multiply by
	Area		
square inch	square centimeter	cm^2	6.4520
square foot	square meter	m^2	0.0929
	Length		
inch	centimeter	cm	2.5400
foot	meter	m	0.3048
mile	kilometer	km	1.6093
	Volume		
cubic inch	cubic centimeter	cm^3	16.3870
cubic foot	cubic meter	m^3	0.02832
gallon	cubic meter	m^3	0.004546
gallon	litter	l	3.7850
	Mass		
pound mass	kilogram	kg	0.4536
slug	kilogram	kg	14.5900

English	SI	SI symbol	To convert from English to SI Multiply by
	Force		
pound	newton	N	4.448
kip (1000 lb)	newton	N	4448
	Density		
pound/cu.ft.	kilogram/cu.mt.	kg/m^3	16.02
pound/cu.ft.	gram/liter	g/l	16.02
	Work, Energy, Heat		
foot-pound	joule	J	1.356
BTU	joule	J	1055
BTU	kilowatt-hour	kWh	0.000293
therm	kilowatt-hour	kWh	29.3
	Pressure		
pound/sq.inch	kilopascal	kPa	6.895
pound/sq.foot	kilopascal	kPa	0.04788
	Temperature		
Fahrenheit	Celsius	C	5/9(F-32)
Fahrenheit	kelvin	K	5/9(F+460)
	Velocity		
foot/second	meter/second	m/s	0.3048
mile/hour	meter/second	m/s	0.4470
mile/hour	kilometer/hour	km/h	1.609
	Acceleration		
$foot/second^2$	$meter/second^2$	m/s^2	0.3048
	Torque		
pound-foot	newton-meter	N-m	1.356
pound-inch	newton-meter	N-m	0.1130