J.K. Cable P.E.

# Bond Development in Concrete Overlays

August, 1995

Sponsored by the
Highway Division of the
Iowa Department of Transportation and the
Highway Research Board

Iowa DOT Project HR-561

Iowa Department of Transportation

report

College of Engineering
Iowa State University

## DISCLAIMER

"The opinions, findings, and conclusions expressed in this publication are those of the author and not necessarily those of the Project Development Division of the Iowa Department of Transportation, nor of the United states Department of Transportation".

J.K. Cable P.E.

# Bond Development in Concrete Overlays

August, 1995

Sponsored by the
Highway Division of the
Iowa Department of Transportation and the
Highway Research Board

Iowa DOT Project HR-561

College of Engineering

Iowa State University

## ABSTRACT

BOND DEVELOPMENT IN CONCRETE OVERLAYS

Data collection to determine the rate of bond strength development between concrete overlays and existing pavements and the evaluation of nondestructive testing methods for determining concrete strength were the objectives of this study.

Maturity meters and pulse velocity meters were employed to determine the rate of flexural strength gain and determine the time for opening of newly constructed pavements to traffic. Maturity measurements appear to provide a less destructive method of testing. Pulse velocity measurements do require care in the preparation of the test wells and operator care in testing. Both devices functioned well under adverse weather and construction conditions and can reduce construction traffic delay decisions.

Deflection testing and strain gaging indicate differences in the reaction of the overlay and existing pavement under grouting versus nongrouted sections. Grouting did enhance the rate of bond development with Type III cement out performing the Type II grout section.

Type III and Type II cement grouts enhanced resistance to cracking in uniformly supported pavements where joints are prepared prior to overlays achieving target flexural strengths.

Torsional and direct shear testing provide additional ways of measuring bond development at different cure times.

Detailed data analysis will be utilized by TRANSTEC, Inc. to develop a bonded overlay construction guidelines report.

## TABLE OF CONTENTS

## LIST OF TABLES



## LIST OF FIGURES

**FIGURE** **Page**

| FIGURE | Page |
|---|---|

**FIGURE** **Page**

## INTRODUCTION

Bonded portland cement concrete overlays have been used in Iowa to extend pavement life for several years. Many questions have been raised over the development and retention of bond between the overlay and the existing pavement. TRANSTEC Inc. and the Texas Transportation Institute (TTI) are under contract with the Federal Highway Administration (FHWA) to study Fast Track paving and its use in thin portland cement concrete (PCC) overlay applications for pavement rehabilitation. This work is also of interest to the Iowa Department of Transportation and the concrete paving industry in Iowa. Previous Iowa research in this area includes a thin overlay of US 71 in Buena Vista County. In another project on I-280, problems developed in loss of bond in selected areas of the overlay. Questions remain as to:

1. The rate of bond development between the new and existing pavement layers.
2. The practical methods for determination of overlay concrete strength and bond with the underlying slab which allows traffic use, and produces desired performance in terms of retarding cracking and bond loss.

## RESEARCH OBJECTIVES

The goal of the TRANSTEC/FHWA research project is the development of a manual to guide others in the practice of placing thin concrete overlays in conjunction with fast track paving as a concrete rehabilitation alternative. The Iowa 3 project in Franklin County provided Iowa an opportunity to assist in the FHWA research effort and increase Iowa's knowledge in the use of nondestructive testing methods for the determination of pavement overlay concrete bond and flexural strengths required for opening to traffic.

The federal study research efforts include the determination of the bond rate development and the relationship between temperature and opening time limitations.

The Iowa Highway Research Board approved a project (HR-561) with Iowa State University and the Civil and Construction Engineering Department to assist TRANSTEC in data collection and to extend Iowa's knowledge in the construction of bonded overlays. Iowa's contribution to the FHWA research included the collection data for the following portions of the field verification of the overlay bond development, retention and strength development as follows:

1. Determination of bond strength rate gain through the measurements of strains, moisture and temperature near the top of the overlay, at the interface between overlay and original slab and at the bottom of the original slab.
2. Development of practical methods that utilize nondestructive testing equipment to monitor rate of flexural strength and bond strength gain for determination of opening to traffic.
3. Determination of appropriate relationships between concrete strength and overlay bond development to allow

use of the total concrete slab structure by traffic without causing potential damage to the overlay.

The TRANSTEC and TTI staff are under contract with FHWA to analyze the data from several sites nationwide and share significant results with the Iowa DOT to further the fast track and bonded overlay innovations in Iowa.

## EXPERIMENTAL DESIGN

### Data Collection Methods

The combined research team of personnel from ISU and TRANSTEC identified several data items for collection on this project. The goals of the national research centered on the measurement of bond development vs construction methods and materials and weather. Data collected was subdivided into the following areas:

1. Structural information - The Iowa DOT Roadrater was used to determine the structural value of the existing pavement prior to overlay, after the overlay and the possible detection of any delamination after construction.
2. Weather data - Two sources were combined to obtain this information. A portable weather station was provided for the early stages of the project by the TRANSTEC staff and located in Hampton, Iowa. Construction field records kept by Iowa DOT staff provided the other portion of the temperature and rainfall records. Temperature and

humidity probes planned for placement in the pavement and subgrade layers were considered by TRANSTEC staff for the project, but not employed.

3. Bond development - The TRANSTEC staff provided the equipment to both monitor bond development in terms of torsional and pullout testing. Bond magnitude after completion of construction was also measured by the Iowa DOT Office of Materials through the use of the Shear Test being performed on pavement cores.
4. Concrete Strain - The TRANSTEC staff provided horizontal strain gages to be placed in longitudinal, transverse, and diagonal positions at the middepth of the overlay. ISU vertically mounted strain gages were employed in longitudinal and transverse locations at one corner of each test slab near the shoulder edge of pavement. In addition, a temperature sensor was placed at the same location to measure the interface temperature during and after construction.
5. Flexural strength - The Iowa DOT chose to test the maturity and pulse velocity concepts for identification of concrete strength and determination of traffic opening times on this project. Maturity meters and a pulse velocity meter were purchased by ISU for this purpose.

**Instrumentation Data Collection Device Selection**

Due to the joint effort nature of this project, many of the data collection instruments were chosen by TRANSTEC staff to provide compatible data with the other sites in the national study. This included the selection of the weather station equipment and the selection of the strain gages to be used for horizontal measurements. The portable weather station provided for this project collected temperature, rainfall, wind and humidity information on a continuous basis. For security reasons it was placed at the temporary staff residence on the north edge of Hampton, Iowa.

The strain gages chosen by TRANSTEC, for the horizontal strain measurements, were vibrating wire strain gages manufactured by ROCTEST Inc.(1). This type of gage is approximately 152.4 mm (6.0 in.) in length and is made up of an embedment device at each end to act as an anchor, a taunt wire between the anchors, an exciter mechanism and appropriate cabling to provide measurement capability outside the pavement shoulder. The device measures of the change in taunt wire length. A 9.14 m (30.0 ft.) cable was attached to each device to provide access for data collection from the roadway shoulder and assure adequate length for placement at any location on the test slab.

TRANSTEC staff also provided their own devices for measuring the bond strength between the overlay and existing slab. In the pullout test, a 76.2 mm (3.0 in.) diameter circular disk and 15.8 mm (5/8 in.) diameter stem were provided for embedment immediately behind the paver. The device was developed to be placed on the existing pavement surface. A 152.4 mm (6.0 in.)

diameter plastic (PVC) cylinder was to be placed in the fresh concrete to allow separation of the test concrete from the remaining portion of the slab.

In the case of the torsional test, a 15.8 mm (5/8 in.) diameter metal stem was welded to four fins that were 101.6 mm (4.0 in.) in length, 25.4 mm (1.0 in.) in height, and 3.2 mm (1/8 in.) in thickness. The arrangement was embedded in the concrete immediately behind the paver, with the bottom of the vanes at the interface between the overlay and existing pavement surfaces. A combination of 101.6 mm (4.0 in.) diameter PVC pipe sections and metal tubing were used to separate the test area from the surrounding concrete. The tubing was greased prior to placement to permit removal after concrete placement. A torque wrench is used to apply a torsional load at prescribed times after concrete placement and measure the force required to break the bond between the new and existing concrete at the interface.

The strain gages employed by ISU staff consisted of two individual model CEA-06-125UN-120 strain gages mounted on opposite sides of a 101.6 mm (4 in.) by 12.7 mm (0.5 in.) strip of 22 gauge galvanized metal. Both gages were mounted on the top 25.4 mm (1.0 in.) of the metal strip. The top 25.4 mm (1.0 in.) was surrounded by a 25.4 mm (1.0 in.) square tube section, 25.4 mm (1.0 in.) in height and secured to the gage post by a section of threaded rod. A 9.14 m (30.0 ft.) length of Beldon lead wire was provided for each strain gage to provide data collection from the road shoulder area.

Temperature data was collected utilizing the TH-1 thermistor prebuilt by ROCTEST Inc.(1). It was purchased with a special data collection device that is capable of recording information from this and the EM-5 gages. It was decided by the research team to place this device near an exterior corner of the existing slab and at the interface between the existing pavement and the overlay.

Maturity was determined using a Model H-2680, System 4101 Concrete Maturity Meter (2) distributed by Humbolt Manufacturing Co. The device provides the potential to monitor four channels or thermocouples simultaneously. The meter can be programmed to begin recording at a user specified time, sample at specified time intervals, record time and temperature, and calculate maturity relationships in terms of time/temperature or equivalent age. It has storage capacity for 365 days of recording and can be downloaded to most popular computer spreadsheets. It was chosen for its programming and recording abilities, number of channels and the ability to readily download the data to user analysis packages. The device is provided with a handy carrying case, thermistors connectors, and cabling for downloading. The device is connected to the pavement by a simple two pole connector and "T" type thermocouple wire. One end of the wire is attached to the connector and the other is stripped for 25.4 mm (1.0 in.) to allow for twisting of the wire ends.

Maturity was also measured in selected locations by the TRANSTEC staff utilizing a portable computer data collector unit.

This unit utilizes the same "T" type thermocouple wire and records the time and temperature relationships through the use of a preprogrammed process control and interface recording device.

The pulse velocity measuring device chosen for this project was recommended by the Iowa DOT Office of Materials staff from previous information and experience of other agencies in its' use. The V-Meter, Mark II is distributed by James Instruments Inc.(3). It includes a carrying case and grease for assisting in making a solid contact between the sensors and the concrete face. This device uses two transducers that must be placed in full contact with the concrete surface and pointed directly at each other. The concrete surfaces are greased to provide full contact with the sensing devices.

### Instrumentation Data Recording Device Selection

In the case of the maturity and pulse velocity measurements, selection of the recording device was first made based on the type, amount and timing of the data to be collected. This dictated the type of recording device to be purchased and in turn the type of gage to be attached.

The selection of the particular custom built vibrating wire strain gage also dictated a particular type of data collection unit. This particular unit allowed the user to obtain the amount of strain and temperature at the center of each gage. It can be used to monitor several types of strain and temperature devices. It can be connected to five separate wires simultaneously to make measurements from a single gage. The MB-6T Vibrating Wire Readout Unit (1), manufactured by ROCTEST, Inc. was also used in this situation to monitor the interface temperature gage.

Strain gage produced by the ISU staff were monitored by a Vishay Strain Indicator, manufactured by Measurements Group Inc. This device provides a digital measurement of strain across the 1/4 bridge arrangement in the strain gage and wiring. Each gage must be read separately, but the reading is obtained in seconds after attachment of the lead wires.

## SITE CHARACTERISTICS

### Test Site Selection

Site selection was determined by the Iowa DOT staff through field and office reviews of potential overlay projects scheduled for construction in 1994. Existing pavement condition was a major factor in the site decision. Pavements in good to fair condition (minimal cracking) were identified to be candidates for the bonded overlay construction. Care was also taken to select a project that could be constructed during the summer months of June through August. This criteria was used to reduce the effect of cool night and daytime temperatures on concrete strength development.

The contract documents specified one lane construction with the use of a pilot car to conduct traffic in the opposing lane. Curing blankets were specified and construction timing was implied in the documents to require concrete placement during morning hours, allow for curing in the afternoon and allow opening the lane to traffic before sunset. The pilot car and lights were to be used until the concrete reached a flexural strength of 2.41 MPa (350.0 psi), transverse joints were constructed and the blankets removed. This requirement placed emphasis on selecting a project with reasonable amounts of traffic (cars and trucks) that could be operated under one or two lane traffic throughout the day and night.

The site selected for this project is a section of Iowa 3 located in Franklin County, Iowa. The construction site selection map is shown in Figure 1. This site has several

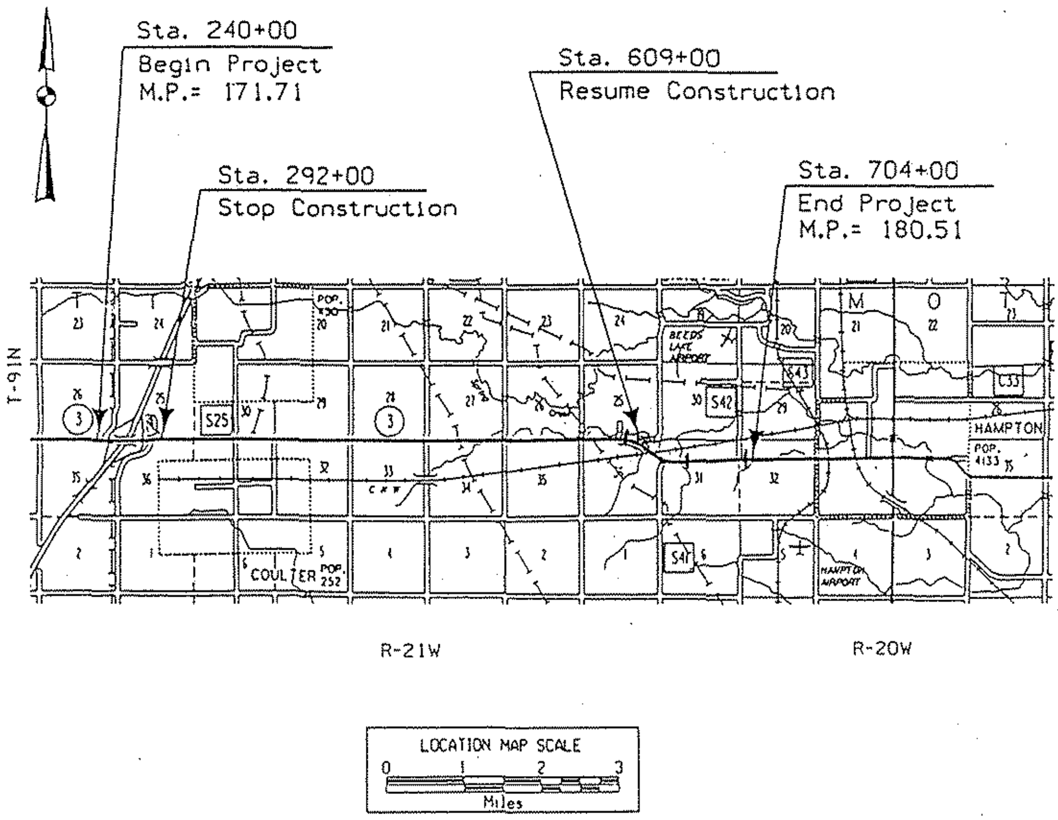



**Figure 1 Construction Project Site Map**

interesting construction implications that may be factors in the performance responses to date. The westerly portion of the project begins approximately 0.80 km (0.5 mi.) west of I-35 and extends easterly 1.61 km (1.0 mi.) over I-35 and past a large truck stop. This portion of the original pavement was constructed in 1963 as part of a relocation of Iowa 3 and the construction of the I-35 interchange. The reconstruction area included several areas of peat removal and replacement with special backfill. In most of the cases this provided adequate support for the new roadway. The pavement was built on a clay subgrade and no special drainage layers or longitudinal subdrains were included. This resulted in pavement faulting and joint deterioration over the life of the pavement. Granular surfaced shoulders, 3.05 m (10.0 ft.) in width, were constructed as part of the original project.

The easterly portion of the project begins 10.46 km (6.5 mi.) east of I-35 and proceeds easterly 2.90 km (1.8 mi.) to a point approximately 1.45 km (0.9 mi.) from the west city limits of Hampton, Iowa. This portion of the pavement was originally constructed in 1931. It consists of a thickened edge cross section that is 254.0 mm (10.0 in.) thick at each edge and 177.8 mm (7.0 in.) at the centerline. It was constructed on a very flat longitudinal grade with minimal ditch depths (approximately 0.9 m (3.0 ft.) below top of pavement). The pavement was also constructed on a compacted earth subgrade without any special drainage layers or longitudinal drains. This section also includes a reverse curve over an abandoned railroad.

In 1969 the pavement in this area was reconstructed. It now consists of a uniform thickness 254.0 mm (10.0 in.) portland cement concrete pavement and 3.1 m (10.0 ft.) wide granular surfaced shoulders.

## Instrumentation Site Preparation

Three sites were selected for instrumentation on the project to measure pavement temperatures, longitudinal, transverse, and vertical strains, and bond development. Each site was selected to be part of the easterly portion of the construction project. Three major factors contributed to the installation in this portion of the project. First, this portion of the existing pavement provided a pavement surface of relatively the same age. The railroad grade separation pavement was built in a different year, but with the same materials as used in the remaining portion of the pavement. Secondly, the project was designed to consider three different bonding situations. The easterly portion of the project provided sections for the use of no grout, and Type II and III cement grouts prior to placement of the overlay. Thirdly, the contractor chose to place the overlay on the easterly portion of the project first. This allowed the research team to gather all the strain and bonding development data in a short amount of time near the beginning of the project.

An individual site was selected for each of the bonding agent situations. Individual slabs were selected that contained no cracking or other visible distresses within the test slab or those slabs immediately adjacent to it in the same or opposing lane. Care was also taken to identify locations where instrumentation wiring could easily be trenched through the shoulder to provide positive drainage for the carrier pipe and easy access for data collection. Initial selection of the sites included slabs in both directions at the following locations:

Site 1. Station 610+50 (Type II cement grout)

Site 2. Station 661+25 (No grout)

Site 3. Station 701+50 (Type III cement grout)

Preliminary work for the research portion of this project consisted of identification of the test slabs, structural testing of the test slabs, laboratory development of the maturity relationships in the laboratory and installation of the various gages.

ROADRATER testing was accomplished by the Iowa DOT, Office of Materials staff. Loads were applied at the center of each test slab (each direction of travel) and near the transverse joint on each end of the test slab, in the outer wheel path. These measurements allow the department staff to develop a structural number for the existing thickness and determine the load transfer capabilities of each transverse joint. Measurements were taken in each lane to provide the option of using either slab and direction for the additional tests.

Samples of the materials to be used in the concrete mix were obtained by the Iowa DOT staff and split for use by both the Iowa project staff and the TRANSTEC/TTI. This required approximately 0.38 $m^3$ (0.5 cy.) of aggregates and the appropriate amount of cement material. This material was stored at the Iowa DOT Central Materials Laboratory in Ames for the testing and splitting. The Iowa portion of the sample was used to cast cylinders and flexural beams in the development of the maturity curves for the project.

Installation of the strain gages proved to be a difficult but not impossible task. The section of roadway to be overlaid in a given day was closed to traffic approximately one hour prior to concrete placement. After closure, the construction area was shotblasted to clean the surface, opened to concrete delivery units for travel and covered with grout immediately in front of the paver. To accommodate this activity, the horizontal strain gage mounting devices (bar chairs) were attached to the pavement with masonry nails, well ahead of the paver. Holes were also drilled in the existing pavement for the vertical type strain gages. Gages were temporarily placed in their locations and wiring was secured to the pavement with duct tape at several locations near the edge of the pavement. The gages were then removed from the slab and placed on the shoulder. Gage wiring was placed in a 50.8 mm (2.0 in.) diameter PVC pipe, trenched 152.4 mm (6.0 in.) deep from the pavement to the outside edge of the pavement shoulder. Each end of the pipe was sealed to prevent intrusion of moisture.

The construction vehicles were guided around the gage chairs to the paver until the paving operation was approximately 15.2 m (50.0 ft.) from the gage site. At this time, the horizontal gages were secured to the chairs with plastic ties, vertical gages were placed in the drilled holes and secured with epoxy and the temperature gage was secured with duct tape on the roadway surface. This was accomplished while the paver continued to advance and concrete was being deposited in front of the paver.

Each instrumentation site can be identified by the existence of the PVC pipe extending through the shoulder and a steel hazard marker identify the location and prevent damage from mowers.

During construction, the maturity meters were employed by the ISU and TRANSTEC staff at the beginning and end of day headers. The pulse velocity meter was used to monitor concrete strength gain at 152.4 m (500.0 ft.) intervals throughout the day's placement.

## CONSTRUCTION SITE ADMINISTRATION

The construction project was under the supervision of the Iowa DOT, Resident Construction Engineer, Arnold Johnson P.E.. The Project Inspector for the Iowa DOT was Mr. David Bergman. Mr. Eric Moody represented TRANSTEC and Jim Cable represented the ISU staff on this project. Field staff assigned to the project for the daily operations by ISU were Jim Riensche and Toby Leitz. Neil Martin from Texas A&M University represented the TRANSTEC field data collection staff. Other personnel from each of the agencies involved were used at various times during the data collection and project construction.

The data collected by the ISU and TRANSTEC staff on the project were shared with the Iowa DOT project inspector as it was collected. In the case of the maturity data, the inspector was present at the time of data collection or was informed of the results by the ISU staff. Iowa DOT staff made decisions on when to open the days' construction to traffic and conducted all direct communications with the contractor, based on the data provided by the ISU staff.

## CONSTRUCTION PROBLEMS

This project was designed and contract documents developed to provide for construction during the warmest months of the construction season (June through August). In this way the maximum benefit of the sun and long periods of warm temperatures could be utilized in the rapid curing and strength gain desired in the maturity concept for acceptance and traffic use. Due to other construction priorities for equipment and personnel, the contractor was unable to meet these dates of construction. Mainline paving on the project actually began on September 7, 1994 and was completed October 8, 1994. September in this portion of Iowa can be unpredictable in terms of weather. In 1994, temperatures were lower than expected during both the day and nighttime hours during the month of September. This reduced the rate of strength gain in the overlay and forced the use of lights and nighttime traffic control. Maturity measuring devices had to be monitored and security provided throughout the night by the ISU staff. It also caused the contractor to work late into the night to open the construction zone to two way traffic and reduced the efficiency of the existing crew during the day. Due to the time of year, weather, and start of schools; replacement and additional contractor staff was not available in this area.

Numerous equipment and material problems at the beginning of the paving operation reduced the rate of paving each day. Initial plans called introduction of the pilot car traffic control at sunrise, paving from sunrise until 11 A.M., curing until maturity was obtained (est. 5-7 hours) and opening the completed area to traffic. Due to the problems noted, this process was abandoned in favor of one that provided for paving during all daylight hours and curing during the nighttime hours. This required the use of the pilot car, site lighting at the ends of the construction zone and night security for the maturity devices.

The contractor was often faced with problems in loss of entrained air, cold temperatures (air and existing slab), equipment breakdowns and personnel shortages. The use of the insulated blankets and the need to both retain them for strength gain and remove them to complete the sawing proved to be very difficult. Cold temperatures, winds and lack of staff made this portion difficult to accomplish at the desired concrete strength. The contractor did not lose focus of the need to complete the project prior to the end of 1994.

Some training of the contractor personnel in the importance of protecting the maturity lead wires from being severed, was necessary. In an effort to reduce this problem, the wires were placed in a 25.4 mm (1.0 in.) PVC pipe across the surface of the shoulder to allow passage of contractor equipment without damage to the wiring or the maturity meter. The meter was placed on the foreslope and sandbagged for protection and security.

In spite of the problems identified, the research work proceeded to completion with cooperation shown between all parties in a difficult environment.

## RESEARCH RESULTS

### Torsional/Pullout Tests

The pullout tests were developed by the TRANSTEC staff to determine the bonding strength at given times during the early life of the overlay. Multiple test units were installed in the pavement. Theoretically, the units should be secured in place on the existing pavement prior to passage of the paver. This proved not to be possible and the units were installed at a given location immediately after the pavement was placed. This meant removal of the concrete, placement of the device and replacement of the concrete and greased separator cylinder. Each device was connected to a hydraulic cylinder and lift was applied in a uniform rate. The maximum pressure was measured at the time of release between the overlay and existing pavement.

In theory, the test appears to measure the desired result. In practice, problems were experienced in the installation of the devices. It was not possible to place the device in front of the paver and retain its position vertically or longitudinally during pavement consolidation. Placement of the device behind the paver resulted in disturbed concrete and the intrusion of concrete between the existing pavement and the metal disk base. This caused significance differences in the results resulting in their removal from further consideration.

The Torsional Shear Test was developed by Eric Moody of TRANSTEC and measured the resistance to turning (torsion) at the overlay/pavement interface. Force is applied to the stem of the device through the use of a torque wrench which measures the force applied. Figure 2 contains the results of the testing completed in the areas employing the Type II and III cement grout. The test was unsuccessful in the No Grout section due to the lack of early bond development. Tests had to be delayed several hours in this area to obtain minimal concrete bond strength and did not provide conclusive results. The data indicates that higher torsional shear strengths can be obtained with the use of the Type III cement grout in the first 24 hours, but also points to a decrease of shear stress after the end of the first day of curing. Additional tests by TRANSTEC in 1995 on other sites may help determine the reason for these results.

The results of the torsional testing illustrate the rate of gain in shear stress over the first 40 hours of curing. In these

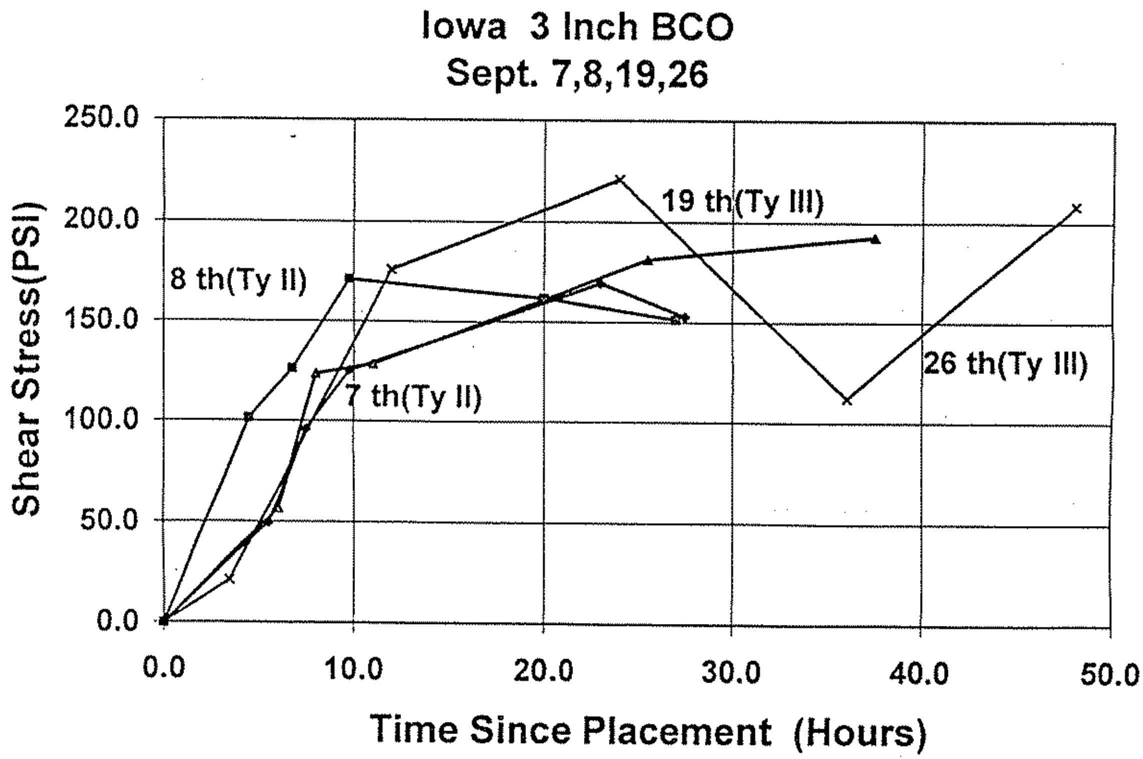



**Figure 2 Torsional Shear Stress Test Results**

test cases there is a noted drop in the shear strain at the end of the first twenty four hours that has not been explained at this time. TRANSTEC staff are conducting additional tests at other locations in 1995 to verify the significance of this data. The Iowa data does indicate a slower rate of gain in shear stress due to the use of the Type III cement grout over that of the Type II grout area, but higher long term shear stress. Both types of grout provided shear stresses in the range of 1.17 to 1.31 MPa (170.0 to 190.0 psi) shear stress values. These values differ from the Iowa shear stress data in terms of the method of test and magnitude of the results. Data regarding the No Grout section is not shown due to the difficulty of obtaining test results in this section.

## Iowa Shear Tests

Some nine cores were obtained by the Iowa DOT staff on December 8, 1994 from the project. They were tested in accordance with Iowa Test Method 406-C, Method of Test for Determining the Shearing Strength of Bonded Concrete (4) by the Iowa DOT Materials Laboratory. This method applies a direct shear load to the interface area of the specimen through movement of two physical surfaces. Cores 101.6 mm (4.0 in.) diameter were removed from the full depth of overlay and existing pavement for testing at random locations. The bond strength is determined by dividing the maximum load applied to the specimen by the cross-sectional area of the specimen. The resulting strengths were obtained from testing completed and reported on December 12, 1994 and are shown in Table 1.

The data indicated one area of weakness near Station 661+65 where no grouting was done as part of the surface preparation. It does indicate the variation that can be obtained from the shear tests conducted. This variation can be attributed to several sources. In this case, variation was introduced in the construction process due to lack of consistent mix in many of the test locations, and the lack of steady forward progress in the paver. The pavement was well cleaned and the grout was applied evenly in the designated areas. Rapidly changing weather

**Table 1, Bond Shear Test Results**

| Core Number | Lab Number | Station Number | Total Load Mg(lbs) | Shear, MPa(psi) | Bond Type |
|---|---|---|---|---|---|
| 1* | ACM4-61 | 267+51 EB,SS | 3.07(6770) | 3.72(540) | Type II |
| 2* | ACM4-62 | 278+99 EB,SS | 3.10(6840) | 3.72(540) | Type II |
| 3** | ACM4-63 | 280+95 EB,SS | 4.18(9210) | 5.03(730) | Type II |
| 4 | ACM4-64 | 661+25 WB,NS | 4.5(9930) | 5.45(790) | No grout |
| 5 | ACM4-65 | 661+45 WB,NS | 1.49(3280) | 1.79(260) | No grout |
| 6 | ACM4-66 | 661+65 WB,NS | 3.52(7770) | 4.27(620) | No grout |
| 7 | ACM4-67 | 701+50 WB,NS | 2.62(5780) | 3.17(460) | Type III |
| 8 | ACM4-68 | 701+20 WB,NS | 2.55(5620) | 3.10(450) | Type III |
| 9 | ACM4-69 | 701+94 WB,NS | 5.48 (12,090) | 6.62(960) | Type III |

Notes: a. * Rain damaged area
b. ** Control
c. EB/WB - east or west bound lane
d. SS/NS - south or north side of pavement

conditions continually changed the curing conditions for the individual areas being tested. Care must be taken to develop a testing plan that will statistically sample the surface and provide adequate sample numbers to account for local variations.

## Horizontal Strain

The vibrating wire strain gages were placed in the positions shown in Figures 2-4 with the vertical strain gages. Data from the gages was plotted in two ways and is shown in Appendix A.

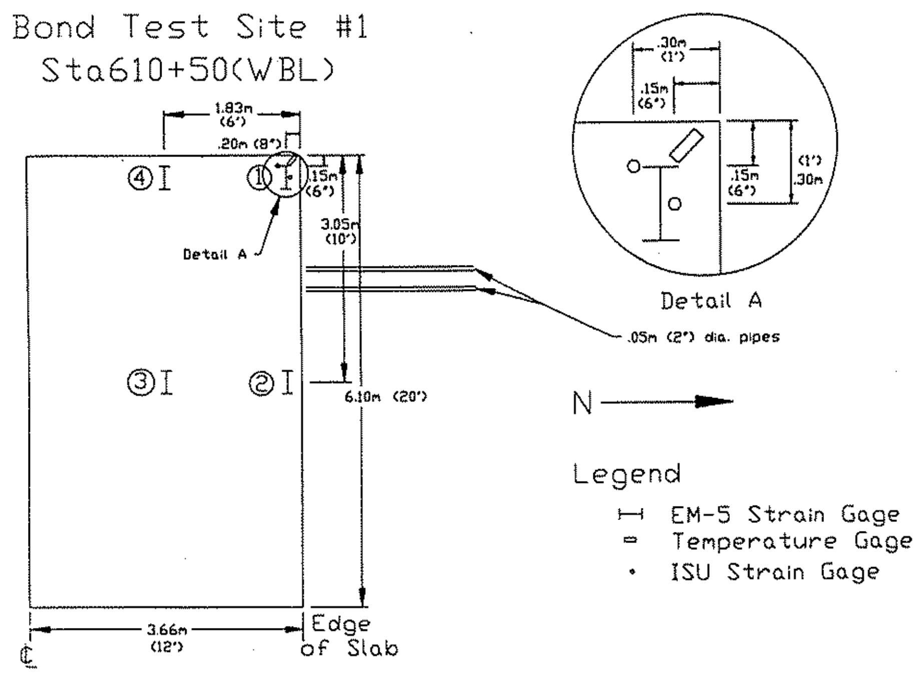



**Figure 3, Site One Instrumentation Layout**

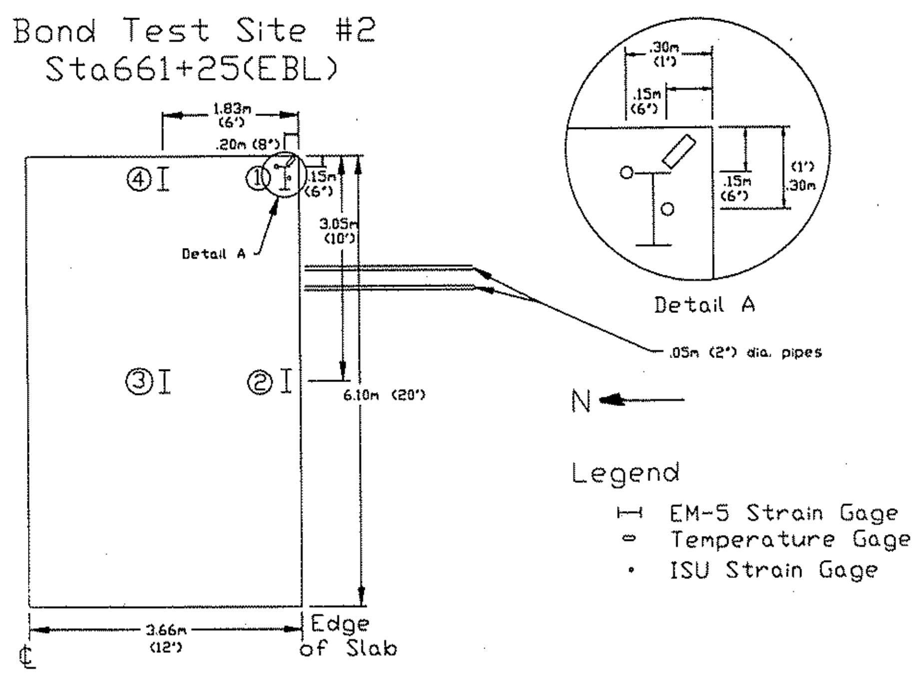



**Figure 4, Site Two Instrumentation Layout**

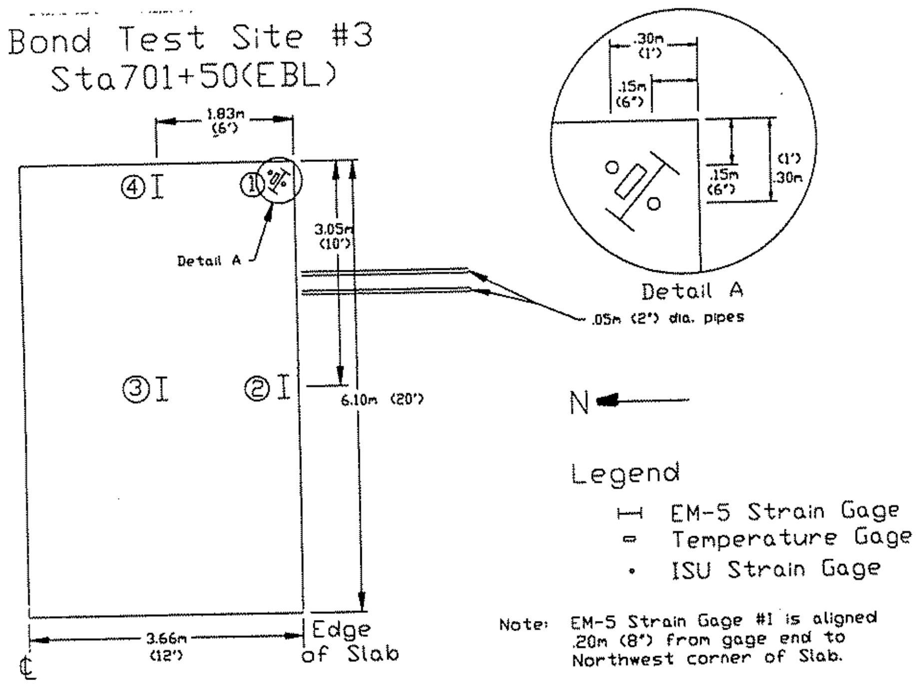



**Figure 5, Site Three Instrumentation Layout**

Information from the gages is first shown in terms of microstrains and temperature over total elapsed time between construction and April, 1995. Separate plots of data obtained in the mornings and afternoons is shown in each graph. Secondly, the information was reduced to that collected over the first 7 or 13 days after construction depending on the date of construction. The reduced data set was analyzed using linear regression for this graph. The graphs provide the following information relative to each test site and gage:

1. Station 610+50 (westbound lane)

    a. Gage 1 indicates a small but increasing compressive strain over time while the temperatures were decreasing during each night and remaining relatively constant during the day. Gage 2 failed to provide data at this site.

b. Gages 3 and 4 indicate higher levels of compressive strain than that of Gage 1, but the trend of the strains and temperature correlate well with that of Gage 1.

2. Station 661+25 (eastbound lane)

a. Gages 1 and 2 indicate increasing levels of tensile strain as the temperatures decreased over time. These levels of tensile strain are more than double that of the compressive strains recorded at Station 610+50 gages.

b. Gages 3 and 4 provide similar data to Gages 1 and 2 at this location. The lack of compressive strains at any time during the day may indicate that the overlay is not bonding fully with the existing pavement and is reacting to the daily temperature changes as an individual pavement layer.

3. Station 701+50 (eastbound lane)

a. Gages 1 and 2 indicate decreasing magnitudes of tensile strain over time as the temperatures indicate large changes from day to night.

b. Gages 3 and 4 show the same general trend as Gages 1 and 2 in terms of decreased tensile strains over time. This trend supports the contention that the overlay is in contact with the existing slab and acting as a composite structure.

## Vertical and Bending Strains

Strain was measured by use of vertical metal strips embedded into the existing concrete pavement. Two strain gages were mounted at the top of each strip. Gage A was mounted on the side of the mounting post away from the joint, Gage B on the side near the joint, Gage C on the side of a second post facing the outside edge of pavement and Gage D being placed on the side of the second post facing toward centerline. The gages were oriented with Micro strains were measured during predetermined times (AM and PM) at each of the gages.

Measurements were made beginning on the day of placement and continuing through the end of the pavement construction period. Additional measurements were made on one day in November of 1994 and one in April of 1995. Data was summarized in terms of the changes in strain experienced by each gage from the initial recorded readings. The data was plotted in two ways and is shown for each site and pair of gages in Appendix B. The first graph for each site and gage pair provides a view of the strain values obtained versus time. In the second case, data for the first 6-7 days of pavement curing was plotted using a linear regression analysis. Linear regression was chosen for the second graph in each case after review of the initial graph results.

The graphs provide the following information for the individual sites:

1. Station 610+50 (westbound lane)
   a. Gages A and B made a strain change between compression and tension over the first 7 days. It supports the idea that the overlay has become part of the total pavement structure and acts as a composite or bonded section.
   b. Gages C and D indicate increasing levels of strain over the first seven days in opposite directions and approximately equal magnitudes. This would also indicate a bonded situation.
2. Station 661+25 (eastbound lanes)
   a. Gages A and B are both indicating increasing levels of tensile strain over the first seven days. This may indicate the potential for loss of bond rather than bending in the vertical test strip.
   b. Gage C failed to record data. Gage D also indicates increasing levels of tension associated with potential loss of bond between the pavement layers.
3. Station 701+50 (eastbound lanes)
   a. Gages A and B show changes between compressive and tensile strains during the day due to the changes in temperature during that period of time. In each case, the predominate shift is toward compression in both gages that would be associated with bonding and the action of the total pavement structure.
   b. Gages C and D also indicate the shift in strains

equal and opposite over time that would be associated with bonding and the action of the overlay as part of the total pavement structure.

**ROADRATER Structural Values**

ROADRATER testing was planned at each of the three instrumented sites to represent pavement structural value and load transfer across transverse joints prior to paving, at one month after paving, and one year after construction to determine the potential for lost bond. The first testing was accomplished on May 9, 1994 prior to paving at each of the test sites and at other selected locations along the project. Testing of the test sites immediately after the construction was suspended due to traffic safety problems. Testing of the overlay was completed on April 6, 1995.

Results of the testing can be viewed in several ways. The structural rating and associated subgrade support value (K) can be used to estimate the value of the overlay in terms of effective thickness of the pavement structure. Table 2 illustrates the results of that analysis. Values shown in the

**Table 2, Deflection Testing Structural Values**

| Test Section | Subgrade Support Value | | Structural Rating | | Effective Depth (mm) Overlay |
|---|---|---|---|---|---|
| | 5/9/94 | 4/6/95 | 5/9/94 | 4/6/95 | |
| 1 | 225 | 188 | 4.94 | 5.94 | 25.4 |
| 2 | 203 | 225 | 4.90 | 7.45 | 129.5 |
| 3 | 225 | 182 | 3.38 | 6.24 | 145.3 |

table for test sections 2 and 3 are located in the eastbound lane where other testing was accomplished. Values for test section 1 were taken in the westbound lane. The cross sectional pavement depth for each of the test sections is unknown, but assumed to be 254.0 mm (10.0 in.). Results shown in the table do indicate an increase in structural value (effective depth) due to the associated nominal overlay thickness of 76.2 mm (3.0 in.). The increase was calculated assuming a structural coefficient of 0.5 per 25.4 mm (1.0 in.) of sound concrete. The results do not directly verify the bonding of the overlay and the existing pavement as noted by the variation in the effective thickness addition. These differences may be the result of variations in cross slope of the existing pavement at each location and longitudinal paving grade at each test site. Site one is very near the end of a transition in grade back to an existing pavement grade. At sites 2 and 3 the nominal overlay thickness was exceeded at the edge of pavement.

The deflection data can also be analyzed in two additional ways. The Iowa ROADRATER has been correlated to the Falling Weight Deflectometer (FWD)(5) in terms of deflections at given locations from the point of loading. The ROADRATER applies a load of approximately 907.2 kg (2,000 lbs, 68 mils at 30 Hz) as compared to the 4082.4 kg (9,000 lb.) FWD load. Structural Capacity of pavements has been estimated using the AREA concept (6). This method measures the structural value in terms of a summation of the deflection ratios (location deflection/

deflection at load) in 304.8 mm (1.0 ft.) increments from the load point to a point 914.4 mm (3.0 ft.) distant. The results are expressed in terms of deflection distance. Using this concept the before and after construction values are shown for each of the test sites in Table 3. A sound portland cement pavement will provide and AREA factor of 29 to 32. The data

**Table 3, Test Section Deflection Area Results**

| Test Section | Calculated AREA(mm) 5/9/94 | 4/6/95 | AREA Change (mm) |
|---|---|---|---|
| 1 | 30.18 | 33.06 | 2.88 |
| 2 | 32.40 | 32.04 | -0.36 |
| 3 | 31.68 | 33.84 | 2.16 |

indicate a potential problem in section 2 which is the nongrouted section. The AREA factor in sections 1 and 3 increases with the addition of the overlay which appears to indicate the existence of a composite section action in each area. In section 2, there is a decrease in the area factor as the result of the overlay. This can be associated with the testing procedures or can be indicating the potential of a delaminated section. This provides one method of identifying potential problems in the future.

An additional use for the deflection data is the analysis of load transfer across the transverse joints. In this testing, the load is placed on the approach side of the joint and deflections are recorded on each side of the joint. The before and after results of this test are shown in Table 4. Load transfer values were computed using the deflection data from the load cells

**Table 4, Load Transfer Results**

| Test Section | Joint Load Transfer (%) 5/9/94 | 4/6/95 | Load Transfer Change (%) |
|---|---|---|---|
| 1 | 92.63 | 65.41 | -27.22 |
| 2 | 84.73 | 86.26 | 1.53 |
| 3 | 100.00+ | 91.86 | -8.14 |

under the load and one 304.8 mm (12.0 in.) away. The percent of load transfer is calculated from the following equation:

%LT = 100(deflection unloaded side of joint/deflection unloaded side of joint)(B)

Where B = Deflection under load ram/deflection at 304.8 mm (12.0 in.) from the ram placed at the center of the slab

Loading is accomplished by placing the ram on the approach side of the joint and testing in the leave direction (traffic direction) across the joint. Information used to calculate the data shown in Table 4 was derived from the average of the two joints adjoining the test site.

The results of this test reflect the fact that the load transfer is actually decreased due to the Iowa DOT policy of sawing the joints in the overlay to full depth. This is done to reduce reflective cracking at the joints. It is interesting to note that site 2 (no grout section) actually indicates a small increase in load transfer. This may be inferring that the overlay is moving independently of the existing slab and locking together to some degree or that the joints in the overlay were not sawed to the full depth of the overlay at this location.

## Maturity Values

Maturity testing was accomplished by the use of two Humbolt System 4101 Concrete Maturity Meters (Model H-2680). Each data acquisition unit is capable of monitoring four channels simultaneously. Samples of the construction materials were used in the Ames, Iowa DOT laboratory to develop a series of cylinders and flexural test beams. These were used in conjunction with the maturity meters to develop theoretical relationships between the mix characteristics and the rate of strength gain under given temperature conditions. The results of this work resulted in a report (2) and graphical/numerical results shown at the end of Appendix C. The results are only applicable to the materials used in this project and separate relationships would be necessary in any of the mix or temperature constraints were changed.

In this project two channels were used to represent the top or surface of the overlay and the middepth of the overlay. A third channel was used in a small number of test locations to collect data about the interface between the overlay and the existing pavement. The meters were numbered to assist in the interpretation of the data obtained. The number one meter monitored the concrete at the beginning of a day's placement and number two was placed near the end of day header. Each meter was placed some 15.24 to 30.48 m (50.0 to 100.0 ft.) from the header to eliminate some of the variation in temperature that occurs with the labor intensive work header construction.

Data from the number two meter was used to determine the time of opening for traffic purposes. The number one meter and the pulse velocity meter were used to help identify the time to begin sawing operations and to gather research data.

Maturity measurements have been identified by Mindess and Young (7) to have certain limitations that may apply to pavement structures and include the following:

1. The measurements do not directly take humidity into account and this cannot be ignored.
2. Measurements are not very useful at low maturities. The time measurements are usually made from the time of placement rather than correctly from the time at which strength gain begins.
3. Maturity is questionable where large temperature variations are present during the curing period. The goal of maturity testing assumes greater strength gain from rising temperatures during the cure period rather than constant temperatures.
4. Strength is affected by the chemical composition and finess of the cement which both affect the rate of hydration. Changes in the cement source or water cement ratio over the course of the project can have an effect on the maturity.
5. The correlation between maturity and strength achieved through accelerated testing relationship breaks down if the specimens are subsequently cooled and moist

cured. Care in preparation of the test specimens is important to the test outcome.

The results of the data collected by each maturity meter were downloaded to a personal computer spreadsheet and used to graphically review the data. The resulting graphs are included in Appendix C. Each graph relates the gain in flexural strength and change in temperature at various depths to the elapsed time from placement of the concrete. Channel one represents the surface of the overlay, channel two the middepth of the overlay and channel three represents the interface between the overlay and the existing slab. The graphs are arranged by data order and time of day. The begin day graphs represent the results from meter number one and the end of day results from meter two. Recorded air and existing pavement (slab) temperatures are also recorded, where available, on the begin day graphs.

The data from the combined beginning and end of day meters provided the following information:

1. Many of the graphs do not indicate that the target strength of 2.41 MPa (350 psi) was achieved. The pavement was not opened prematurely. The experimental time of contact for each meter was limited to approximately 24 hours due to the number of meters available. Using only two meters, it was necessary to move them each day to their new location and stop data collection at the previous location. In many of the cases shown where low strengths appear to be the final

result, the meter was removed for security purposes, and the pavement was allowed additional cure time due to a holiday or weekend without traffic use. Pulse velocity meter data collected at intermediate points 152.4 m (500.0 ft.) intervals and that data does verify the presence of the target flexural strength prior to opening to traffic. In some cases, one of the meters was used at more than one location in a given day and is represented by two begin or end of day graphs.

Further analysis indicates that the use of the datum temperature of +11 degrees versus -10 celsius (established in the laboratory testing and preprogrammed in the meters) accounts for the difference (1 MPa) in the meter values versus those calculated and the pulse velocity values. Future work should utilize the -10 degrees celsius for datum temperature.

2. Combinations of flat concrete strength values and zero temperatures for a given channel indicate that the thermocouple wire was damaged or removed and is no longer valid.
3. Concrete delivery temperatures are available, but tended to remain in the 26 to 32 C (mid 80's F) and were not plotted. Air and slab temperature were of interest due to the fall season and the presence of cold winds on the project. This had an effect on the resulting rate of strength gain. In the case of morning concrete placement

on cold pavements with windy conditions, the loss of heat reduced the rate of gain. As the existing slab warmed to the end of day, the results of meter two indicate a much faster rate of gain in strength. Pulse velocity results also indicate that warming of the base or existing pavement accelerates the strength gain as would be expected.

Data from September 17 also illustrates the problems of reduced strength gain when temperatures begin to fall after 10 hours. Surface temperatures fluctuated over the 24 hour period, while the mid-depth temperature only dropped. The effect of temperature at both locations was a reduction in the rate of strength gain.

4. Temperatures at both the surface and mid-depth levels tend to peak at approximately 7 hours after placement resulting in a reduction on both temperature and strength gain after that point in the curing. This is consistent with the laboratory results. It also indicates the importance of weather on the curing process during the early stages of curing for achieving desired strengths.
5. Concrete placed during the period of October 5-8, 1994 represents much of the miscellaneous work associated with the interchange ramps and median concrete. Monitoring by maturity meter was terminated when no personnel was on the site for security reasons. The concrete was covered and not opened to traffic until it had achieved a time of

cure in excess of that required in the previous work done on this project.

## Pulse Velocity Values

Pulse velocity measurements were made at intermediate locations on days when the concrete placement exceeded 152.4 m (500.0 ft.). Laboratory correlations between velocity, temperature and flexural strength were used to determine the field values required to meet the required flexural strength for traffic opening. These relationships are included in the graphs at the end of Appendix C.

The results of the field work applications are shown graphically in Appendix D. Two graphs were plotted for each location which is specified in the subheading. The first relates temperature near the mid-depth of the overlay and flexural strength to the elapsed time since concrete placement. The second graph utilizes pulse velocity values in place of the temperature relationship with the strength and time. The location of each test site is identified in the subheading and by date in legend.

The following conclusions can be drawn from the information presented in the graphs:

1. Graphs for September 7 through September 30 concrete placement indicate temperatures between 5 and 10 degrees cooler than the temperatures recorded by the maturity meters during the first 7 hours. They also indicate

falling temperatures over the course of the measurements. Insulation boards were placed into the measurement wells between readings to protect the surfaces and loss of heat.

2. Pulse velocity values were obtained for four sites on October 1 and 5, but only the temperatures and resulting flexural strengths were recorded. The temperature/ strength graphs have been included.
3. Acceptable flexural strength were attained in 5.5 to 9 hours in the majority of cases. The rate of strength gain often paralleled the gains in temperature. As temperatures decreased, the rate of gain was reduced, but not eliminated.
4. September 7 pulse velocity values indicate the difficulty in obtaining consistent readings with this device. This data was used to indicate the frequency of testing done on subsequent days.

## Distress Survey Results

The pavement was visually inspected upon completion of the paving and during the following spring. The walking survey was completed by walking along the shoulder of the roadway, beginning at the west end of the project and proceeding easterly. The individual distress types were identified using the "Distress Identification Manual for Long-Term Pavement Performance Project, SHRP-P-338" (8). In this project, transverse, longitudinal, and corner cracking, and joint spalling were the predominate types of distress that were considered. The use of the insulating blankets created the potential for several footprints in the pavement surface that are not normally found in pavement surfaces.

The first survey was conducted on October 7 and 9, 1994 and the second was conducted on April 15, 1995. The results of each survey are included in the Tables 5-6.

**Table 5, Visual Distress Survey Results, October 17, 1994**

| Distress Type | Type II Grout (No. of cracks) | No Grout (No. of slabs) | Type III Grout (No. of cracks) | Total Distresses (No.) |
|---|---|---|---|---|
| Transverse Cracking | 51 | 7 | 2 | 60 |
| Corner Cracking | 1 | 0 | 0 | 1 |
| Transverse Joint Opening | 2 | 0 | 0 | 2 |
| Transverse Joint Spalling | 8 | 0 | 0 | 8 |
| Footprints | 7 | 0 | 0 | 7 |

The tables indicate a growth in the number of transverse cracks and corner cracks while the joint openings, joint spalls and number of footprints remained the same. The large growth in cracking came in the areas where Type II grout had been applied (west portion of the project) and in the no grout section. Transverse cracking increased from 51 to 58 (14%) cracks and corner cracking from 0 to 6 (600%) cracks between surveys in the

**Table 6, Visual Distress Survey Results, April 15, 1995**

| Distress Type | Type II Grout (No. of cracks) | No Grout (No. of slabs) | Type III Grout (No. of cracks) | Total Distresses (No.) |
|---|---|---|---|---|
| Transverse Cracking | 68 | 17 | 2 | 87 |
| Corner Cracking | 11 | 2 | 1 | 14 |
| Transverse Joint Opening | 2 | 0 | 0 | 2 |
| Transverse Joint Spalling | 8 | 0 | 0 | 8 |
| Footprints | 8 | 0 | 0 | 8 |

I-35 interchange area. In the other portion of the project utilizing the Type II grout, transverse cracking increased from 0 to 10 (1000%) cracking and corner cracking from 1 to 5 (500%) cracks between surveys. No change was noted in the Type III grout area.

## Crack Reinforcement

The Iowa DOT Office of Materials also conducted tests on the potential use of reinforcement to retard reflective cracking of midslab cracks that appeared to be nonworking cracks. Some 42 locations were identified as test sites. This included both control and test locations. Deformed, epoxy coated, number 5 bars, 0.9 m (36.0 in.) in length, were placed across each test crack on 0.5 m ( 1.5 ft.) center to center spacings. Each bar was strapped and nailed to the existing pavement immediately in front of the paving operation. Tests were made on cracks in each of the surface preparation type areas.

Each of the test sites was visually inspected as part of the visual distress survey process. Results are shown in Table 7.

**Table 7, Crack Location and Repair Results**

| Crack Number | Station | Direction Length | Direction, length | 10/7/94 Survey | 4/15/95 Survey |
|---|---|---|---|---|---|
| **Station 609, Type II Grout** | | | | | |
| 1 | 610+12 | WB, 12 | | NONE | NONE |
| 2 | 612+50 | EB, 12 | | NONE | NONE |
| 3 | 613+62 | | WB, 20 | NONE | NONE |
| 4 | 613+82 | | WB, 20 | NONE | NONE |
| 5 | 614+70 | WB, 12 | | NONE | NONE |
| 6 | 615+26 | EB, 12 | | NONE | NONE |
| 7 | 615+52 | WB, 12 | | NONE | NONE |
| 8 | 615+68 | EB, 12 | | NONE | NONE |
| 9 | 620+35 | WB, 12 | | NONE | NONE |
| 10 | 621+70 | EB, 12 | | NONE | NONE |
| 11 | 635+68 | EB, NB | | NONE | NONE |
| 12 | 640+48 | EB, NB | | NONE | NONE |
| **Station 645 to 675, No Grout** | | | | | |
| 13 | 646+49 | EB, NB | | CRACK | CRACK |
| 14 | 647+08 | EB, NB | | CRACK | CRACK |
| 15 | 653+04 | EB, NB<br>WB, NB | | CRACK | CRACK |
| 16 | 661+80P | EB, 12 | | NONE | NONE |
| 17 | 661+98P | EB, 12 | | NONE | CRACK |
| 18 | 662+39P | EB, 12 | | NONE | NONE |
| 19 | 662+59P | EB, 12 | | CRACK | CRACK |

| | | | | | |
|---|---|---|---|---|---|
| 20 | 662+81P | EB, 12 | | CRACK | CRACK |
| 21 | 662+97P | EB, 12 | | NONE | NONE |
| 22 | 663+90 | | WB, 20 | NONE | NONE |
| 23 | 663+98P | EB, 12 | | NONE | NONE |
| 24 | 667+75 | WB/EB, 12 | | NONE | NONE |
| 25 | 667+85 | WB, 12 | | NONE | NONE |
| 26 | 668+20 | WB/EB, 12 | | NONE | NONE |
| 27 | 669+38P | EB, 12 | | | NONE |
| 28 | 669+39P | WB, 12 | | | NONE |
| 29 | 669+57P | WB/EB, 12 | | | NONE |
| 30 | 669+80P | WB, 12 | | NONE | NONE |
| 31 | 669+99 | WB, 12 | | NONE | NONE |
| 32 | 670+18 | EB, 12 | | NONE | NONE |
| 33 | 670+41 | EB, 12 | | NONE | NONE |
| 34 | 670+60 | EB, 12 | | NONE | CRACK |
| 35 | 672+56 | EB, 12 | | NONE | NONE |
| 36 | 672+78 | WB, 12 | | NONE | NONE |
| **Station 675 to 704, Type III Grout** | | | | | |
| 37 | 675+74 | WB, NB | | NONE | NONE |
| 38 | 675+91 | EB, NB | | NONE | NONE |
| 39 | 682+28 | WB, 12 | | NONE | NONE |
| 40 | 684+48 | EB, 12 | | NONE | NONE |
| 41 | 686+91 | EB, 12 | | NONE | NONE |
| 42 | 702+12 | WB/EB, 12 | | NONE | NONE |

Notes: a. NB indicates no bars placed

b. 12 indicates length of crack in feet

c. P indicates crack in existing patch

The data indicates that the process was successful in the case of the Type II grout situation with no reflective cracks appearing to date. The grout and bonding in this section may also be the reason for no cracks reflecting in the control areas. All the control cracks in the no grout section cracked within the first 30 days after construction. Four (19%) of the remaining cracks in this section cracked between construction and the April 15, 1995 survey date. Each crack is hairline in size and is being held relatively tight with the addition of the reinforcement. The increased level of reflective cracking may be associated with the independent action of the existing slab and overlay or lack of adequate bond. Section three using the Type III grout provided the same performance as the section employing Type II grout. No reflective cracking was found in this section in either of the surveys or in the control cracks.

## Flexural/Compressive Strength Testing

Test specimens were cast at the time of concrete placement at each of the test sites. Flexural beams were cast and retained at the test sites for testing and 2, 4, 8, 12 and 24 hour tests. Cylinders were transported to the Ames, Iowa DOT laboratory for testing. The results of the flexural beam tests are shown graphically in Appendix E. The flexural test results indicate that each of the preparation methods produced the same long term strengths, but that the Type III grout produced the quickest strength gain to allow for early opening to traffic. A summary of the compressive strengths (cylinders) and flexural strengths (test beams) is shown in Table 8.

**Table 8, Test Speciman Strengths**

| Test Site | Compressive Strengths MPa(psi) 7 day | 28 day | Flexural Strength MPa(psi) 7 day | 28 day |
|---|---|---|---|---|
| 1 | 35.03 (5080) | 41.61 (6035) | 6.00 (870) | 6.34 (920) |
| 2 | 43.11 (5652) | 42.64 (6184) | 6.34 (920) | 6.48 (940) |
| 3 | 35.59 (5162) | 44.56 (6462) | 6.27 (910) | 6.27 (910) |

## SUMMARY AND CONCLUSIONS

The objectives of this study centered on the collection of data to determine the rate and amount of bond that could be developed in the early stages of concrete curing and the evaluation of nondestructive testing methods to be employed for reducing traffic delays. Each of the test methods employed on this project pointed to the same conclusions.

The shear test is currently employed by the Iowa DOT as a method of determining bond strengths after the construction of the pavement overlay. In this case it indicated that the final shear stress (several days after construction) was very similar in magnitude regardless of the type of cement grout employed to enhance the bonding. The No Grout section provided mixed results and indicated the potential for lack of bond. This section did develop bond, but not in the time frame (less than 10 hours) that is desired for traffic control on this project. The torsional shear test represents a way to measure shear stress at an early concrete age. In this case it indicates that the use of the Type III cement grout does enhance the rate of bond strength gain and allow for earlier traffic opening.

TRANSTEC staff will be providing the final analysis of the horizontal strain (vibrating wire gages) data. It can be noted from the Iowa data that there are differences in the type of strain (tensile or compressive) and the magnitude of the strain varies by site and preparation grout type. Sites 1 and 3 employing the Type II and III grout provide time traces that would indicate the overlay is in contact with the existing pavement and responding to the changes in temperature during the day and night. The same information indicates much larger magnitudes of tensile strain in the Type III grout area than the Type II area. This will bear further analysis with regard to existing pavement cross section at each location and the long term effects of the higher tensile strains on long term performance.

The ISU gaging system results shown in Appendix B. Gages A and B are mounted to identify movement in a longitudinal direction. Each gage contains an initial strain due its construction and the configuration of the mounting device in the concrete. If bending occurs, the two gages should indicate opposite strains (tensile and compressive) at any given point in the day or night. This is illustrated in the Site 1 (Station 610+50) and Site 3 (Station 701+50) data. This would relate to the composite action of the overlay and underlying pavement undergoing curling stresses during the cool mornings and hot afternoons. Site 2 (Station 661+25) data exhibits only tensile strains that are increasing over time, both during cool morning hours and hot afternoon hours. It represents the activity associated with a very small temperature gradient (thin section) or overlay versus the activity that would be associated with the composite section. It appears to indicate that full bond may not be occurring at the location of the sensors. Gages C and D are oriented in a transverse direction and should represent similar strains as A and B. Strain levels are larger in the transverse direction at Site 1, but display the same relationship as the longitudinal gages. Strains at Site 3, gages C and D illustrate a similar relationship for gage C. Gage D at Site

3 may be malfunctioning in that there is no change over time. Site 2 gages C and D illustrate the same problem as noted in gages A and B and the potential for loss of bond at this location.

The ROADRATER information gathered from the project was analyzed in several ways. A review of the structural ratings before and after construction related to the thickness of the overlay when an AASHTO coefficient of 0.5 per 25.4 mm (1.0 in.) of sound concrete. The data does indicate an increase in total structural value, but would indicate overlay depths of 25.4 mm (1.0 in.) to 145.3 mm (5.72 in.). These values appear more reasonable when to those who observed the original pavement and its many slabs that were tilted in various directions. This uneven profile and surface cross section created many variations in the actual overlay depths.

Using the deflections at all the sensors the area of the deflection basins can be compared to that of sound concrete. Values obtained at each test site relate to the desired total value, but those obtained at Site 2 show a small decrease in structural value after the overlay construction. This may be indicating the loss of bond or partial bond at this location.

When the deflection data is used to consider changes in load transfer across the transverse joints, the bond situation is again identified. If the overlay is sawed full depth and full bond occurs, the load transfer value should decrease. This happened in Sites 1 and 3. At Site 2 an increase in load transfer was noted that may indicate the overlay is moving free of the underlying pavement and joints are locking due to the surface temperatures.

Nondestructive testing was accomplished by use of the maturity and pulse velocity meters. Each device was laboratory calibrated to the project mix. The results of both meters were comparable when the datum temperature of -10 degrees Celsius is used. The maturity values obtained in the field did relate follow the laboratory created curves. Target strengths of 2.41 MPa were achieved when blankets were readily applied and maintained on the pavement and temperatures were rising during the day. Surface and middepth sensors can be used to assist in the determination of when to begin sawing operations. Cold morning slab temperatures versus warm midday and afternoon temperatures caused the overlay to reach target strengths at different times. It indicates that the sawing sequence should be tied to slab and air temperatures and not to the beginning of the days' placement. Curing time and maturity values were effected by the loss of the blanket protection and decreasing temperatures. This often meant required times of 10 to 20 hours rather than 7 to 10 hours as demonstrated in the laboratory. Peak temperatures in the concrete overlay were noted at 7 to 10 hours.

Pulse velocity measurements appear to be providing correct data when full contact is obtained with the concrete surface. This requires operator training and skill in the development of the test holes in the concrete. Maintaining a representative temperature and moisture contents in each test hole wall at the middepth, between test readings, is difficult to impossible. The results indicate similar relationships to that achieved with the maturity meters.

Distress surveys indicate that the majority of cracking can be categorized as transverse and corner types. A major portion of the transverse cracking is related to the time of joint sawing and the underlying subgrade condition. Unstable subgrades in the westerly portion of the project coupled with improper timing of joint sawing accounted for the bulk of the cracks. The corner cracking appears to be associated with the placement of the shoulder stone while the concrete was not fully cured. Cracking in the No Grout section has more than doubled in the first six months of performance. This may also be an indication of the lack of bond between the overlay and existing pavement. The use of the Type III grout appears to have contributed to the reduced number of reflective and joint cracks.

Reinforcement of the existing midpanel cracks with the addition of reinforcing bars proved helpful in the grouted sections. All cracks have occurred in the No Grout prepared section. Future research may consider locating such reinforcement at the middepth location of the overlay to assure full bonding with the concrete.

## REFERENCES


1. **Vibrating Wire Readout Unit Model: MB-6t Instructional Manual**, IRAD Gage Division of ROCTEST

2. **Maturity and Pulse Velocity for Early Strength Development of Concrete Pavement**, C. Ouyang, Iowa Department of Transportation, 1994.

3. **V-Meter Instructional Manual, Mark II**, James Instruments Inc. Nondestructive Testing Systems

4. **Method of Test for Determining the Shearing Strength of Bonded Concrete**, Test Method No. Iowa 406-C, Iowa Department of Transportation, December 1991.

5. **Correlation of the Road Rater and the Dynatest Falling Weight Deflectometer**, final Report for MLR-91-4, K. Jones, T. Hanson, Iowa Department of Transportation, July 1991.

6. **Backcalculation of Asphalt Concrete-Overlaid Portland Cement Concrete Pavement Layer Moduli**, Transportation Research Record 1293, Transportation Research Board, 1991, pp. 112-123.

7. **Concrete**, S. Mindess and J.F.Young, Prentice-Hall, Inc., Englewood, California, 1981.

8. **Distress Identification Manual for the Long-Term Pavement Performance Project**, Strategic Highway Research Program, National Research Council, SHRP-P-338, Washington, D.C., 1993.

9. **Fast-Track Concrete Paving: More Than Just High-Early Strength**, L. W. Cole, G. F.Voight, Concrete International, May, 1995.

10. **Time to Rein in the Flexure Test**, O.Riley, Concrete International, June, 1995.

11. **Use of Maturity and Pulse Velocity Techniques to Predict Strength Gain of Rapid Concrete Pavement Repairs During the Curing Period**, P.A.Okamoto and D.Whiting, Construction Technology Laboratories, Inc.

12. **Guidelines for Timing Contraction Joint Sawing and Earliest Loading for Concrete Pavements, Volume I: Final Report**, FHWA-RD-91-079, P.A.Okamoto, P.J.Nussbaum, K.D.Smith, M.I.Darter, T.P.Wilson, C.L.Wu, S.D.Tayabji, Construction Technologies Laboratories, Inc., February, 1994.

## APPENDIX

A - LONGITUDINAL STRAIN MEASUREMENT RESULTS

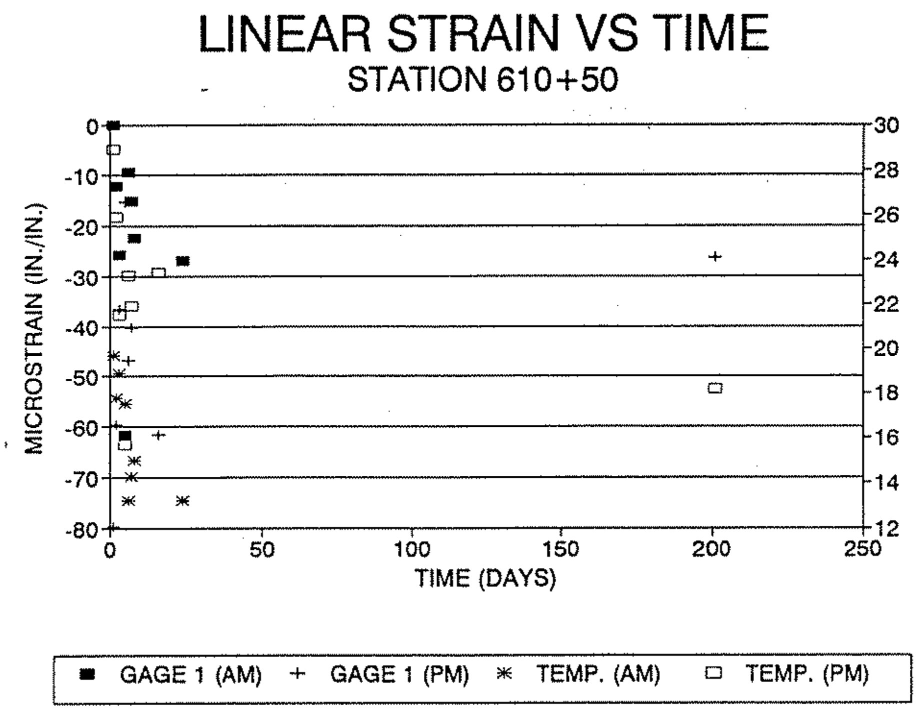
B - VERTICAL AND BENDING STRAIN MEASUREMENT RESULTS

C - RECORDED MATURITY MEASUREMENT RESULTS

D - RECORDED PULSE VELOCITY MEASUREMENT RESULTS

E - FIELD FLEXURAL STRENGTH TEST BEAM RESULTS

## APPENDIX A



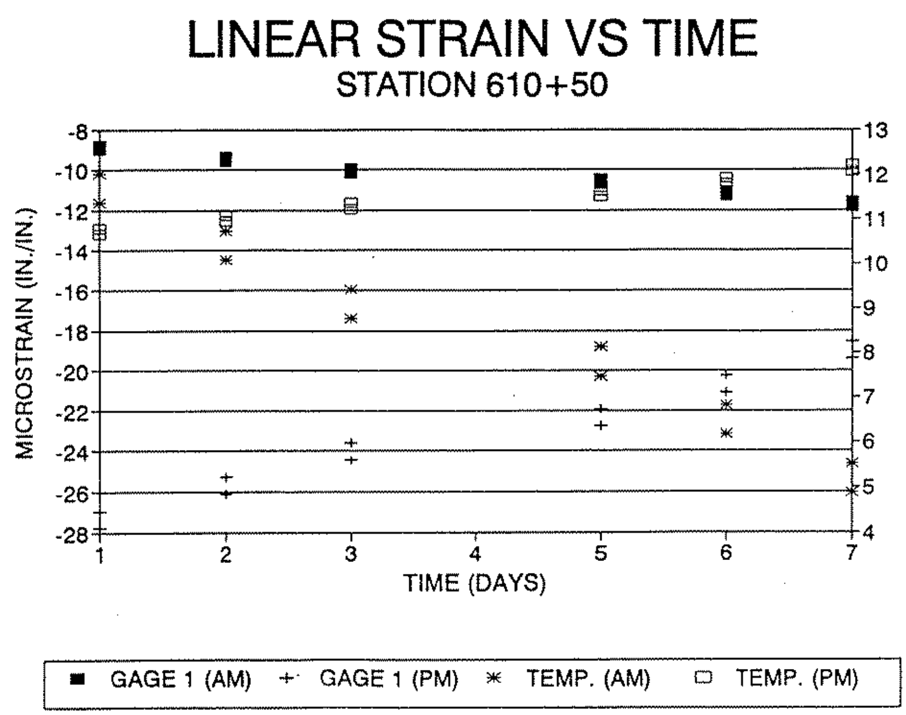
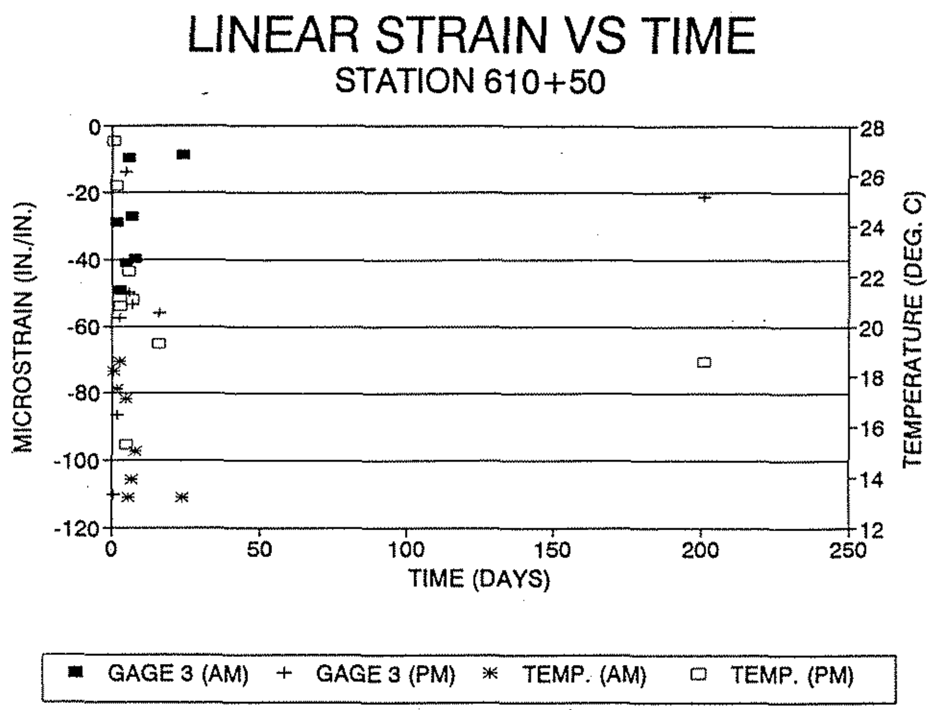




Figure A-1 Station 610+50, Gage One Strain



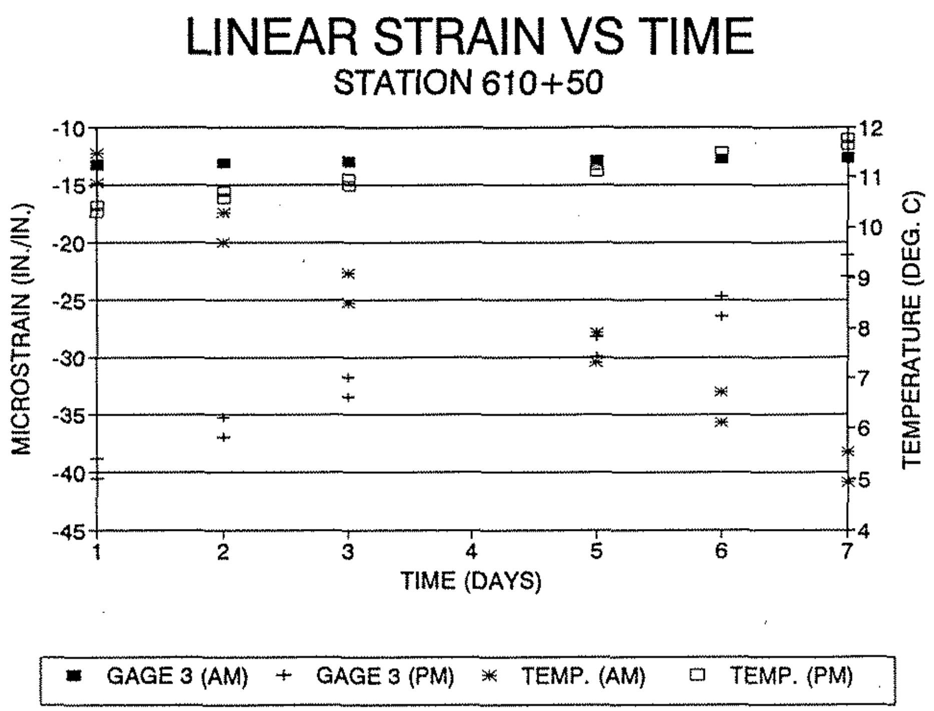
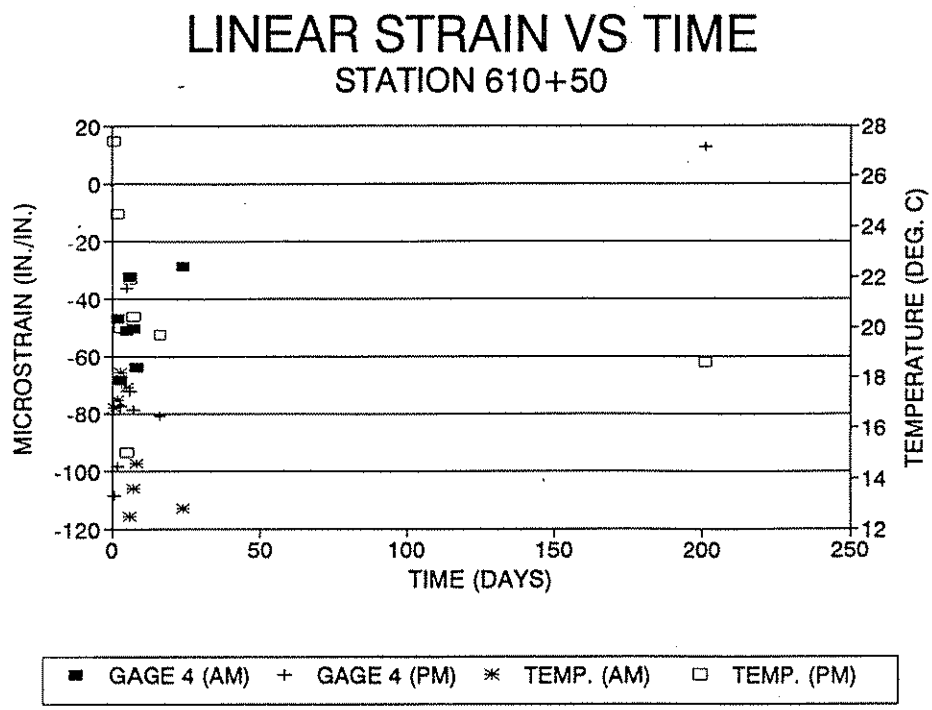




Figure A-2 Station 610+50, Gage Three Strain



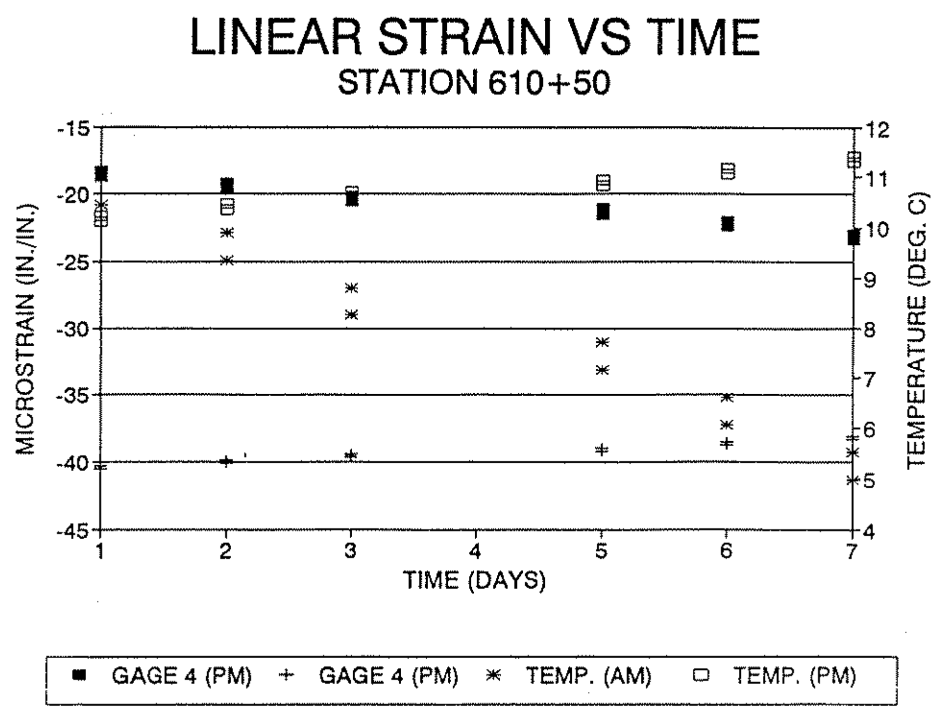
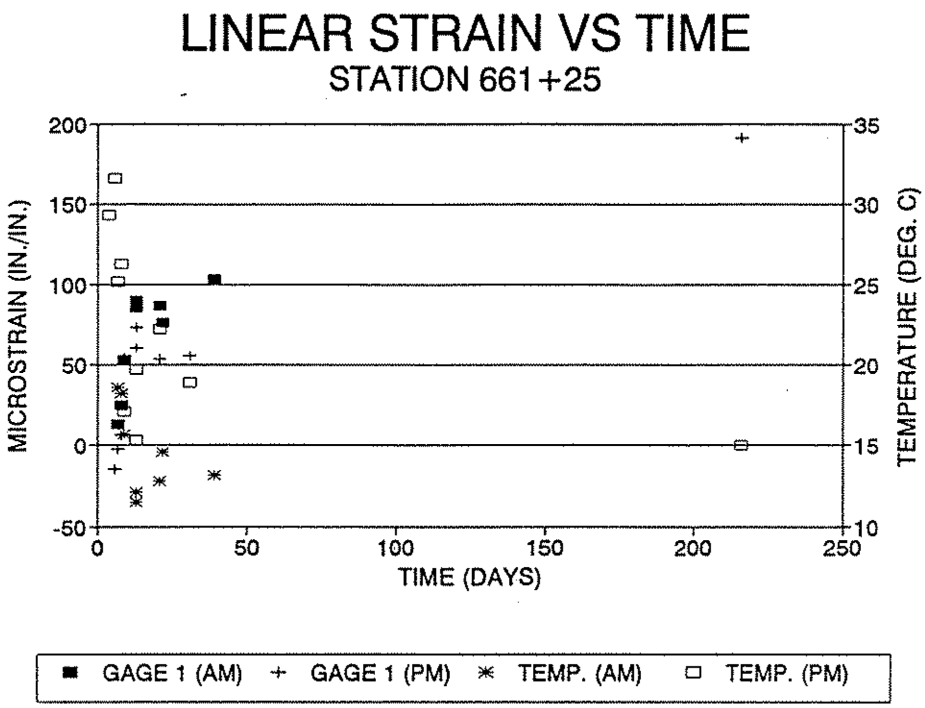




Figure A-3 Station 610+50, Gage Four Strain



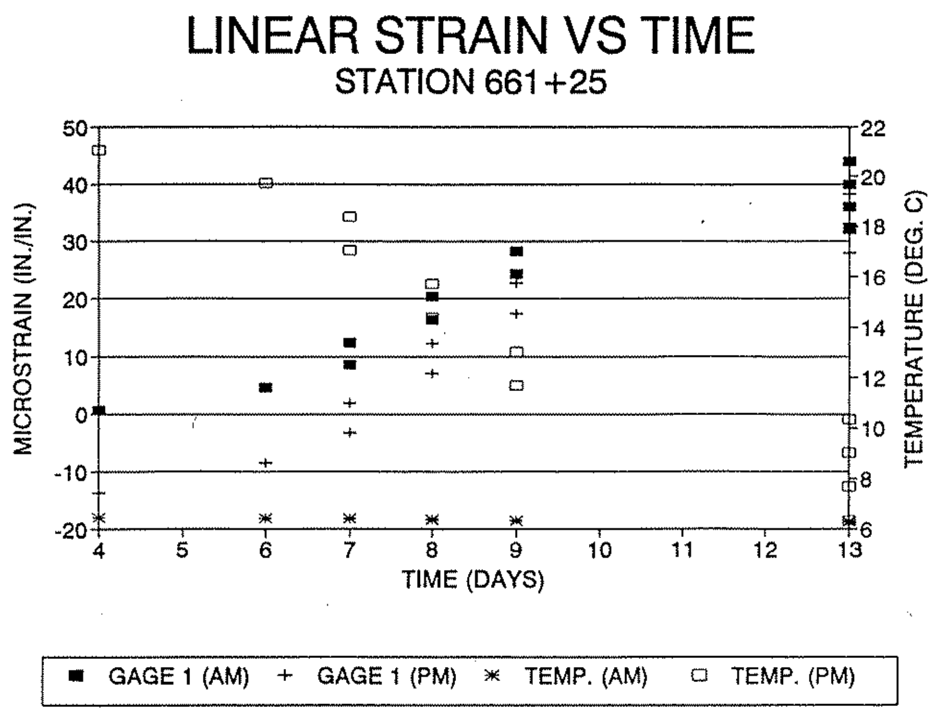
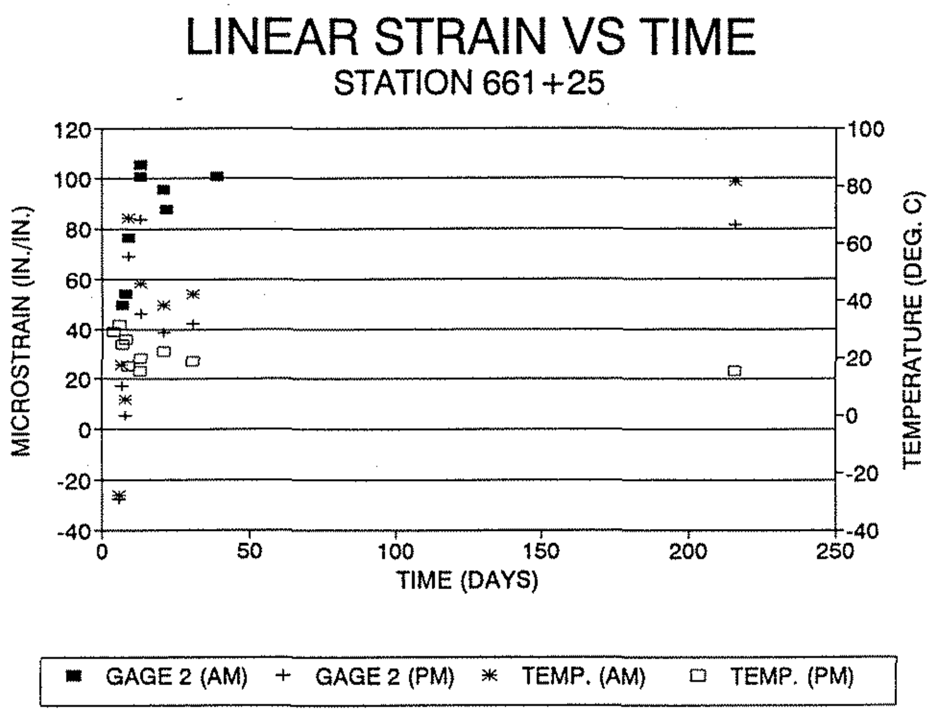




Figure A-4 Station 661+25, Gage One Strain



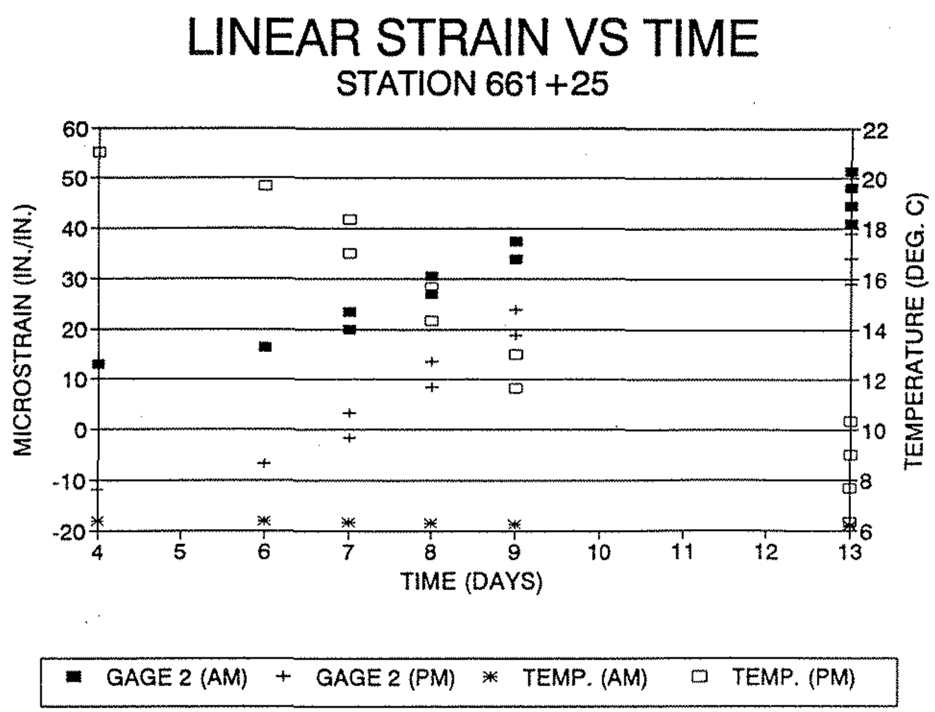
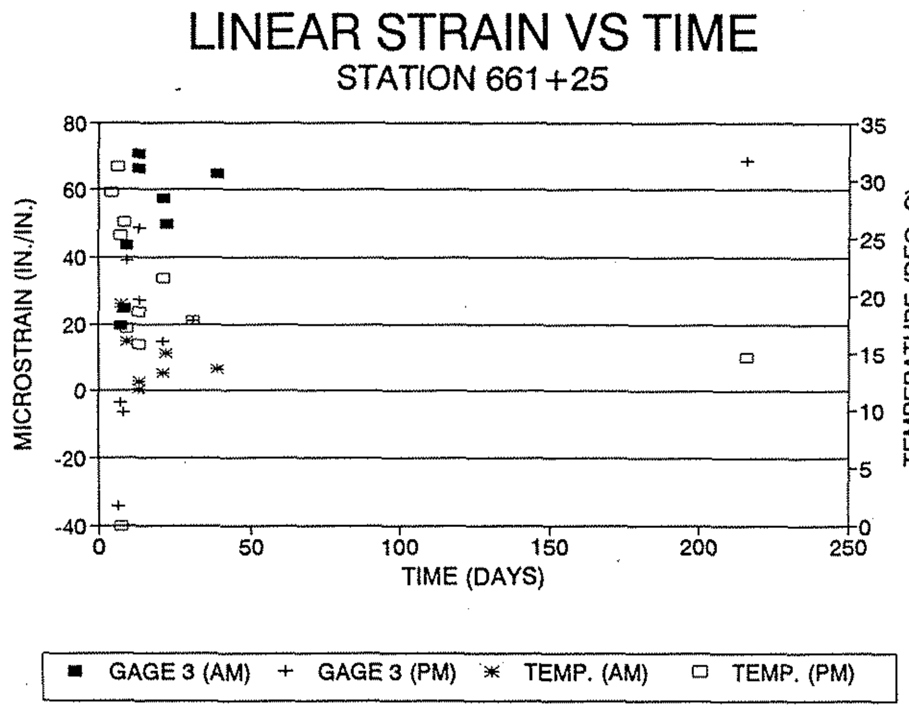




Figure A-5 Station 661+25, Gage Two Strain



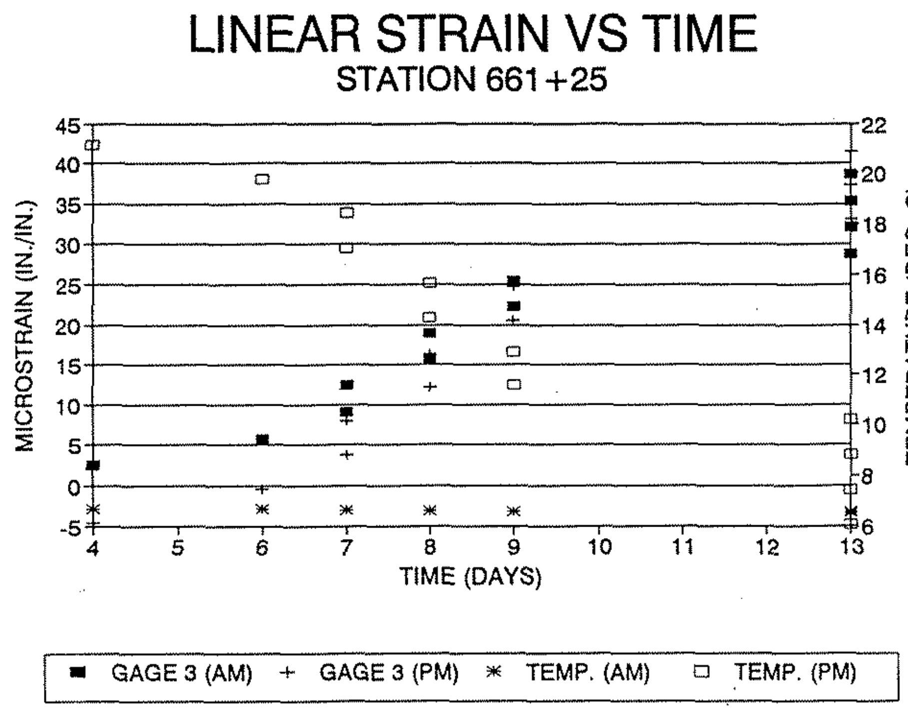
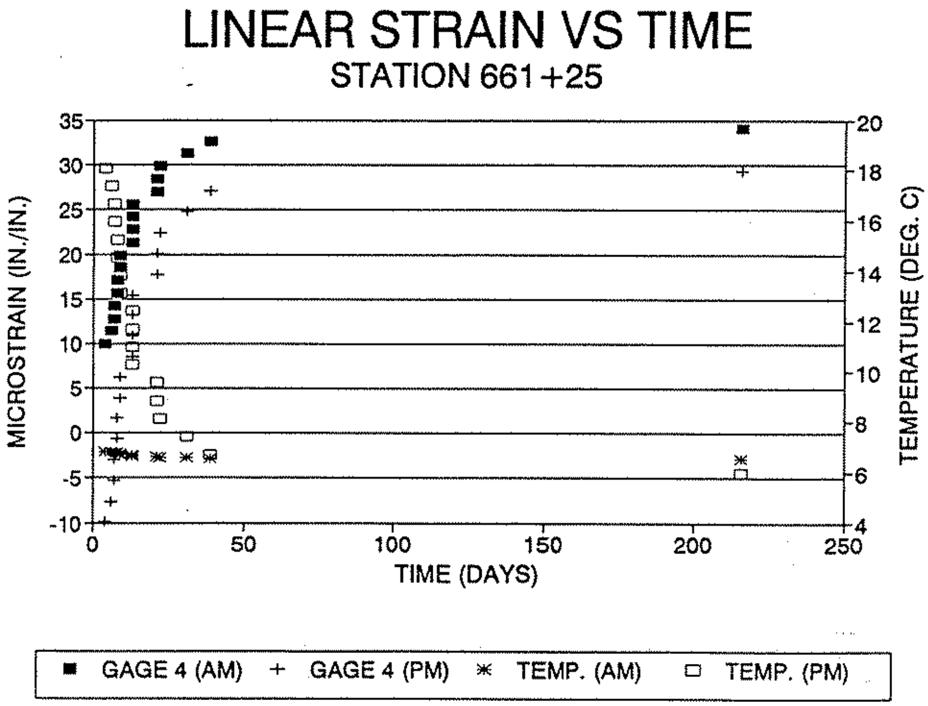




Figure A-6 Station 661+25, Gage Three Strain



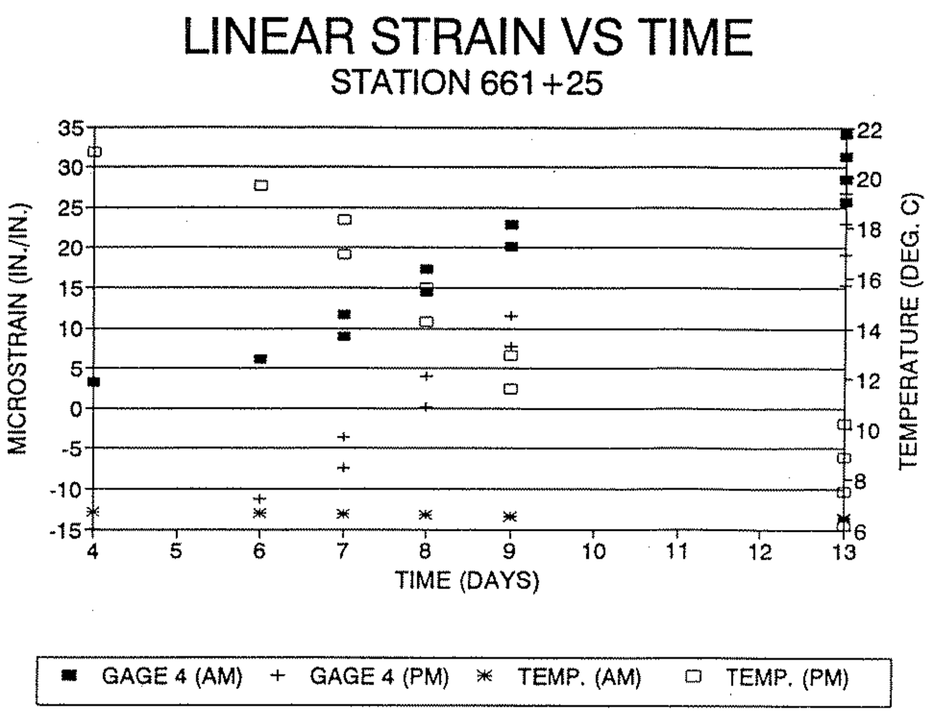
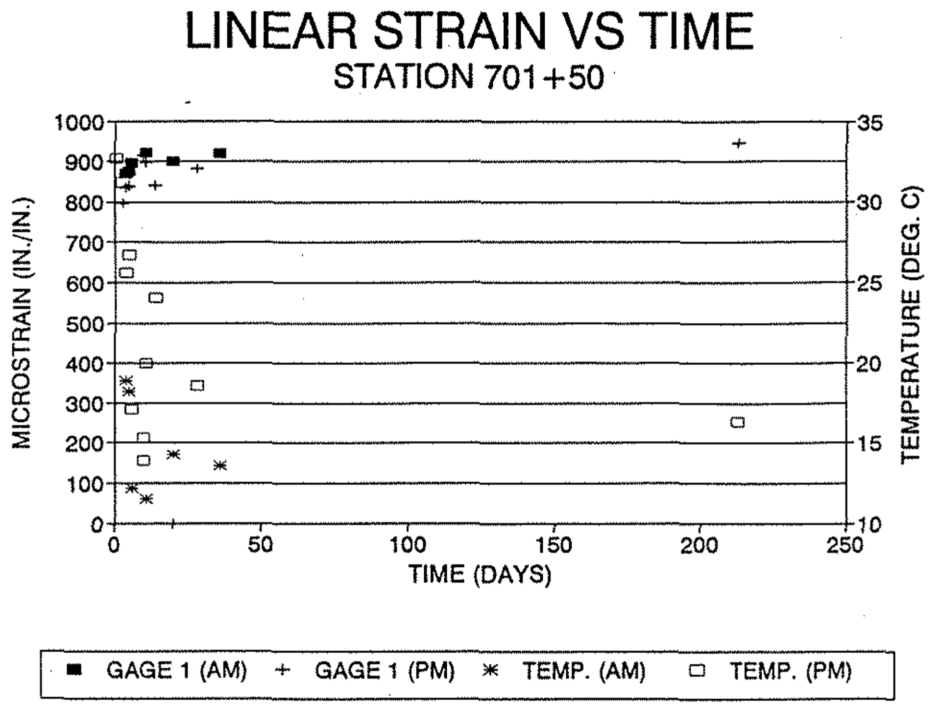




Figure A-7 Station 661+25, Gage Four Strain



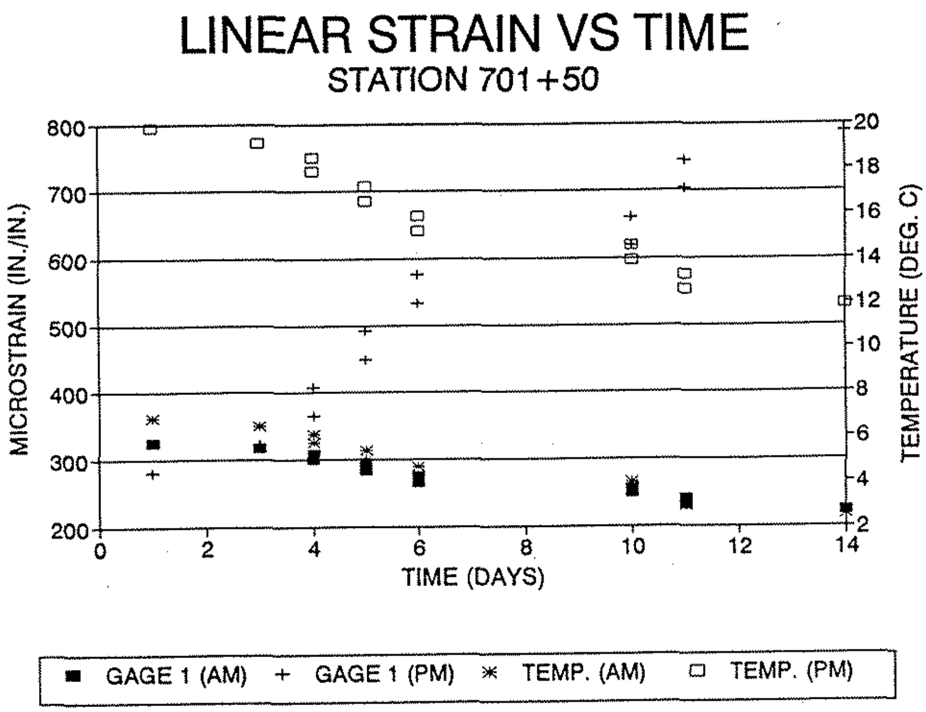
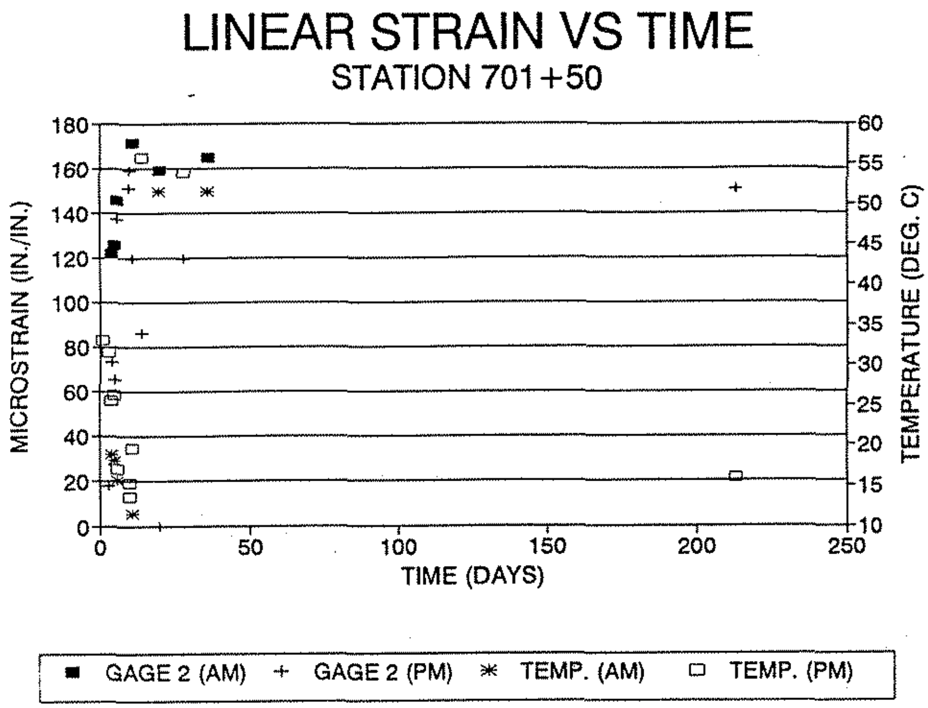




Figure A-8 Station 701+50, Gage One Strain



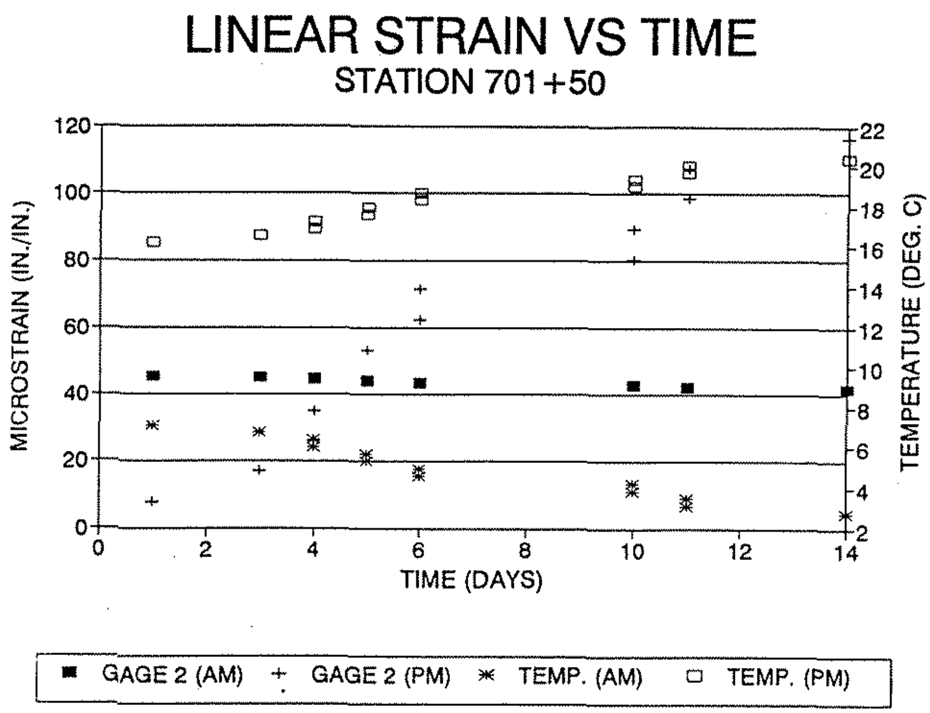
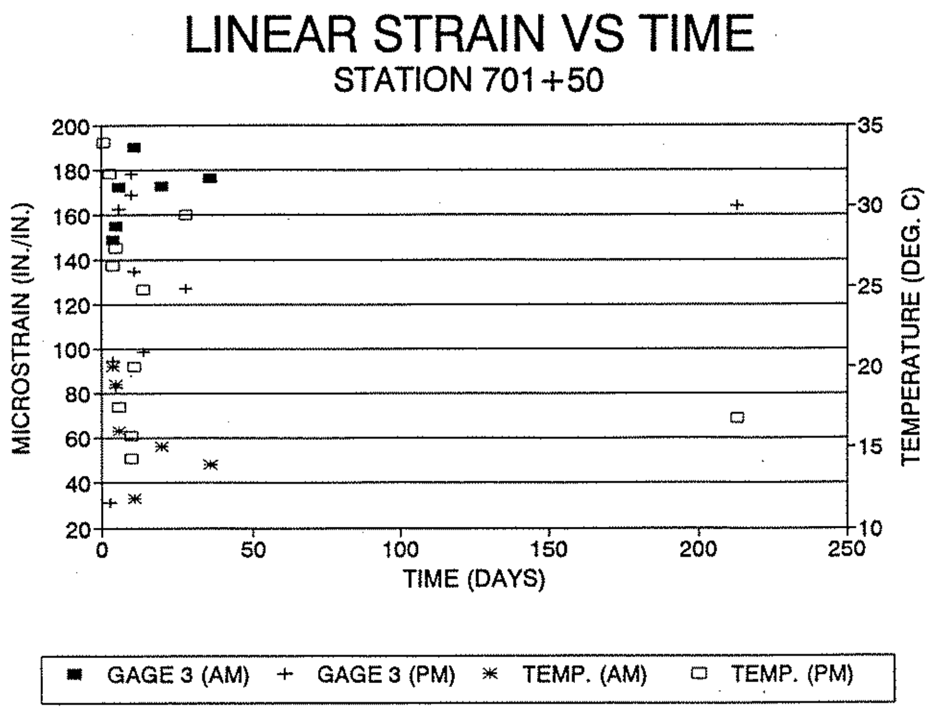




Figure A-9 Station 701+50, Gage Two Strain



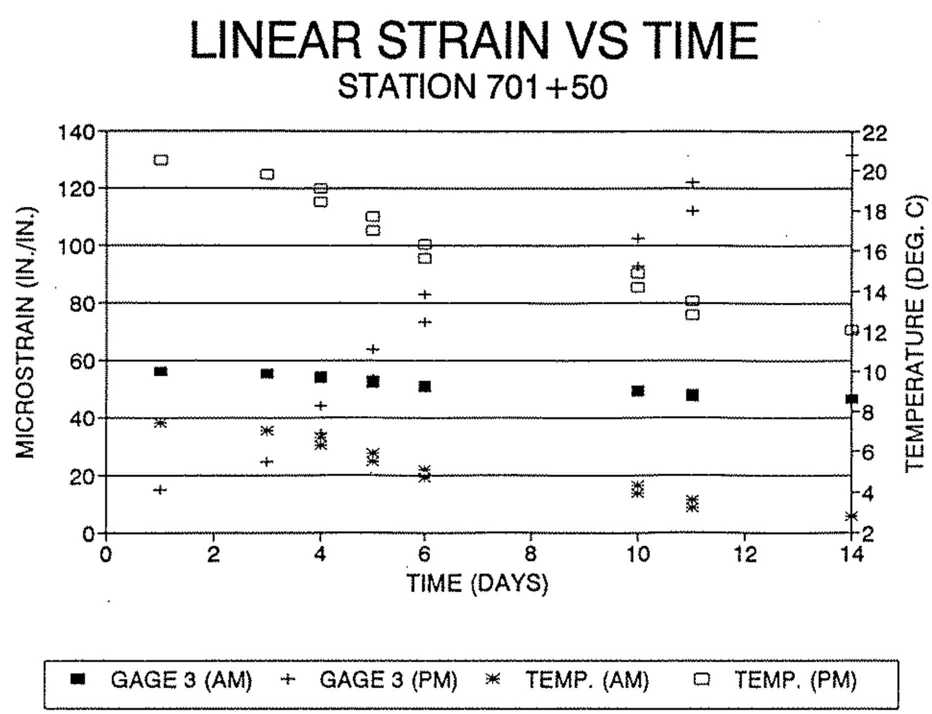
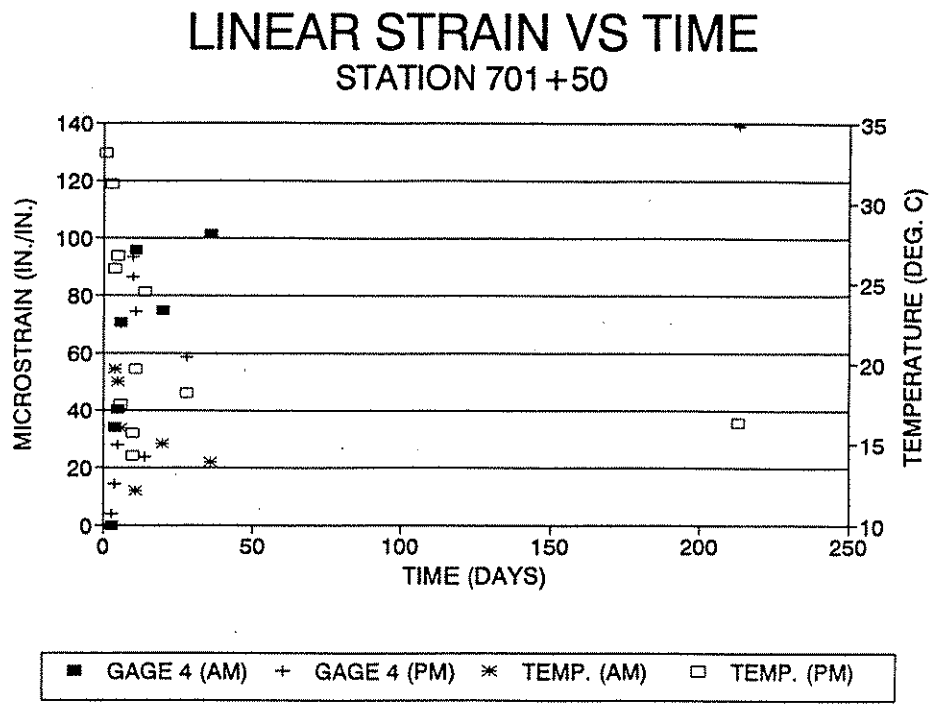




Figure A-10 Station 701+50, Gage Three Strain



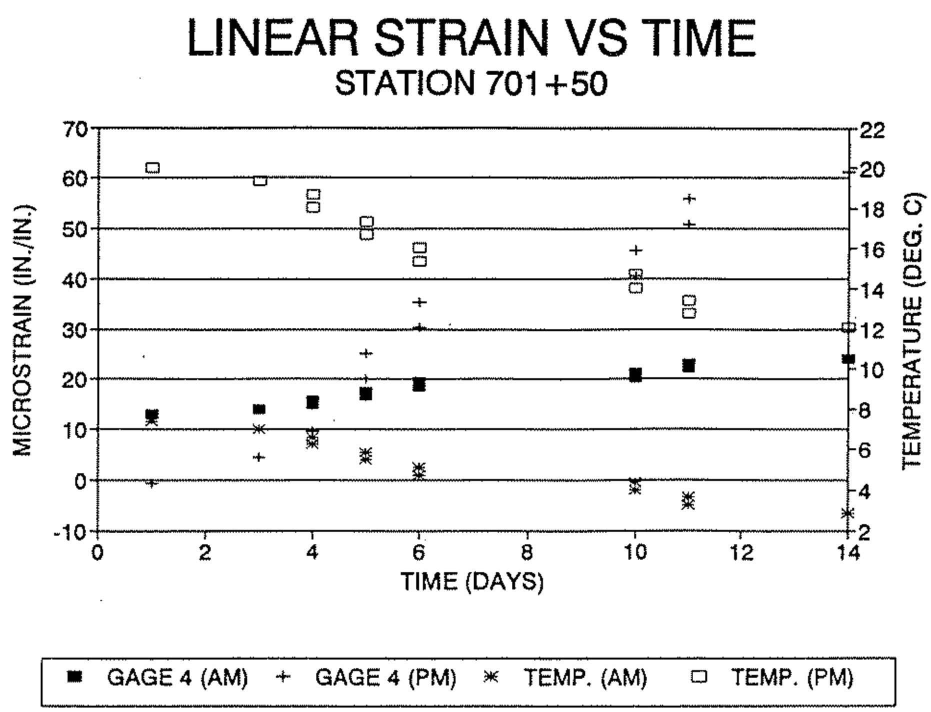
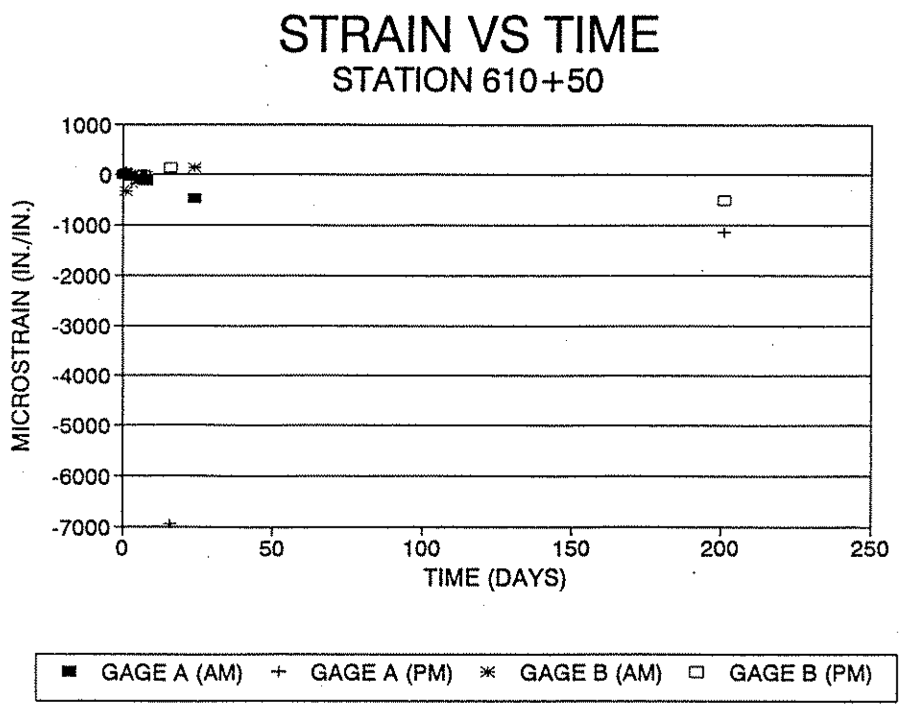




Figure A-11 Station 701+50, Gage Four Strain

## APPENDIX B



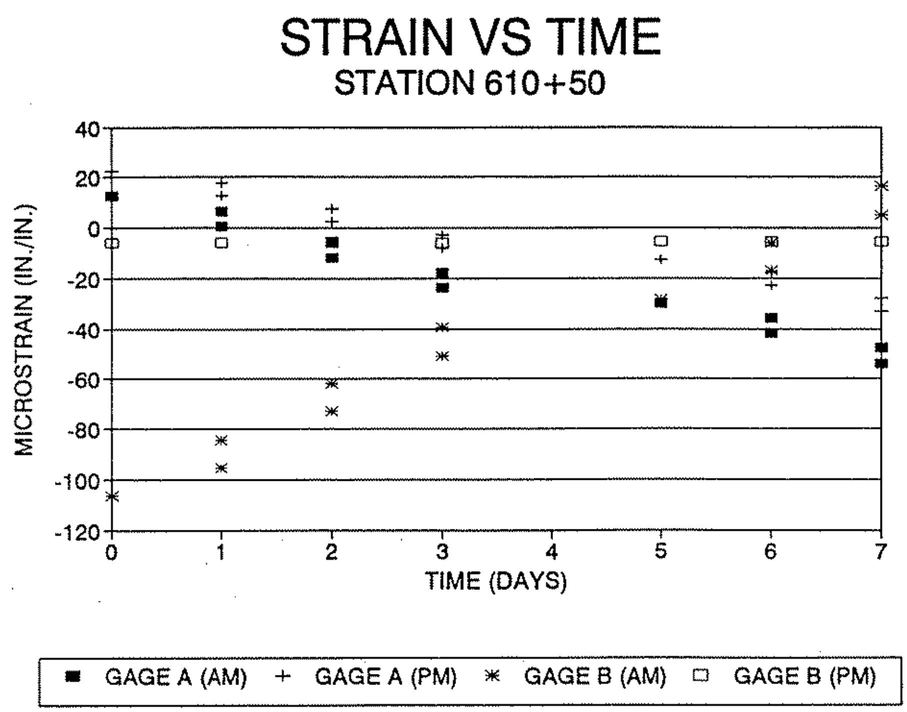
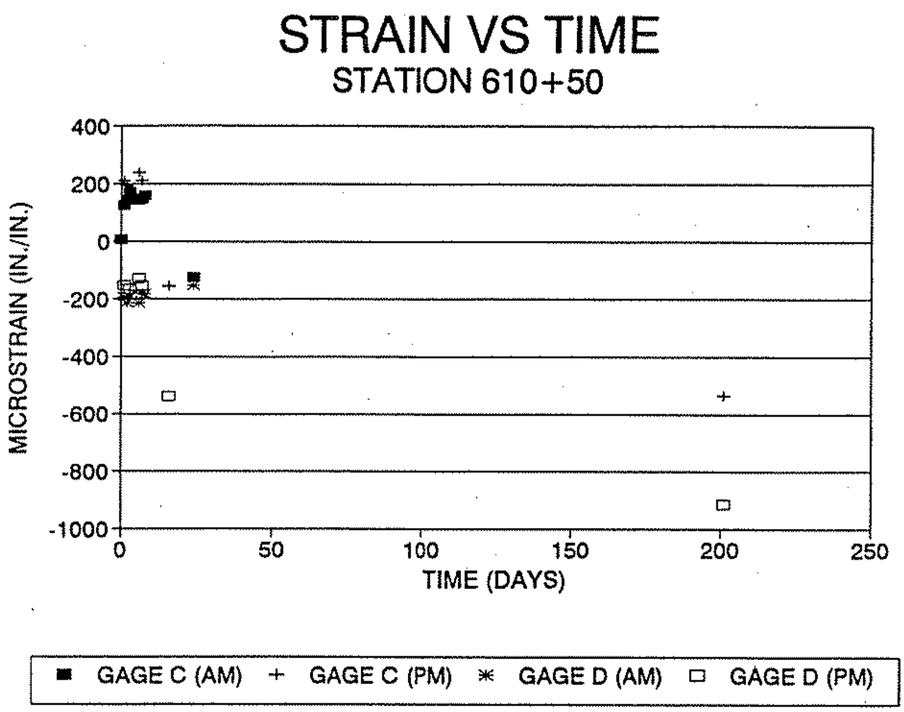




Figure B-1 Station 610+50, Gages A & B Strain



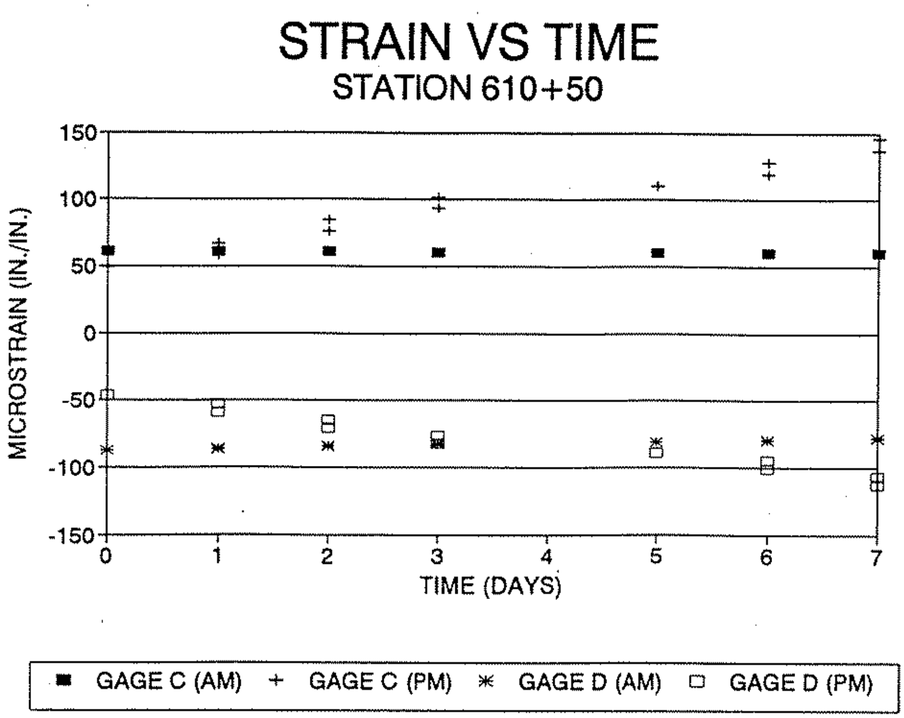
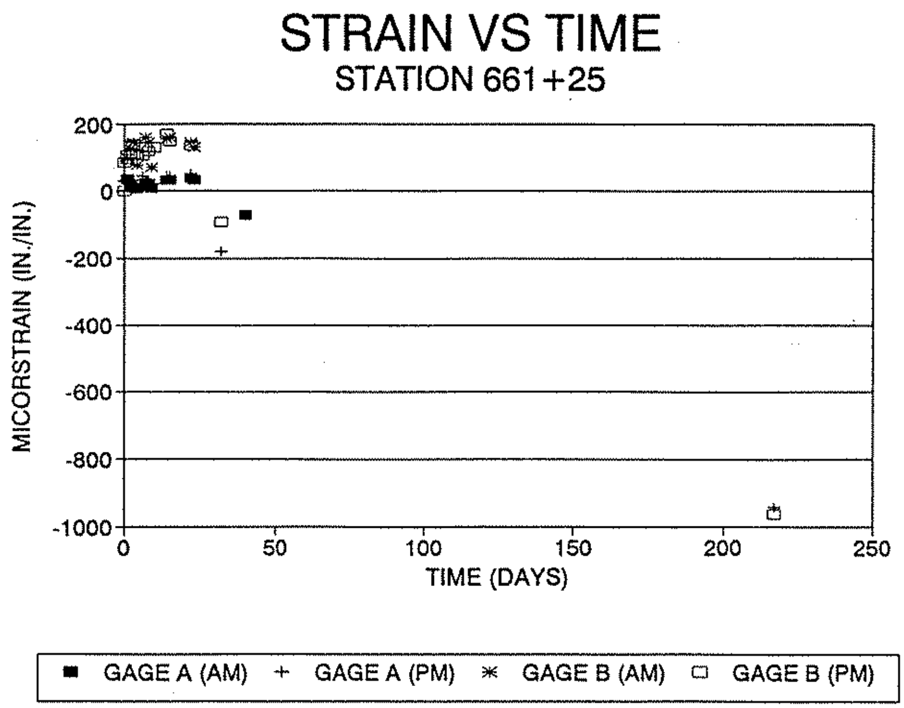




Figure B-2 Station 610+50, Gages C & D Strain



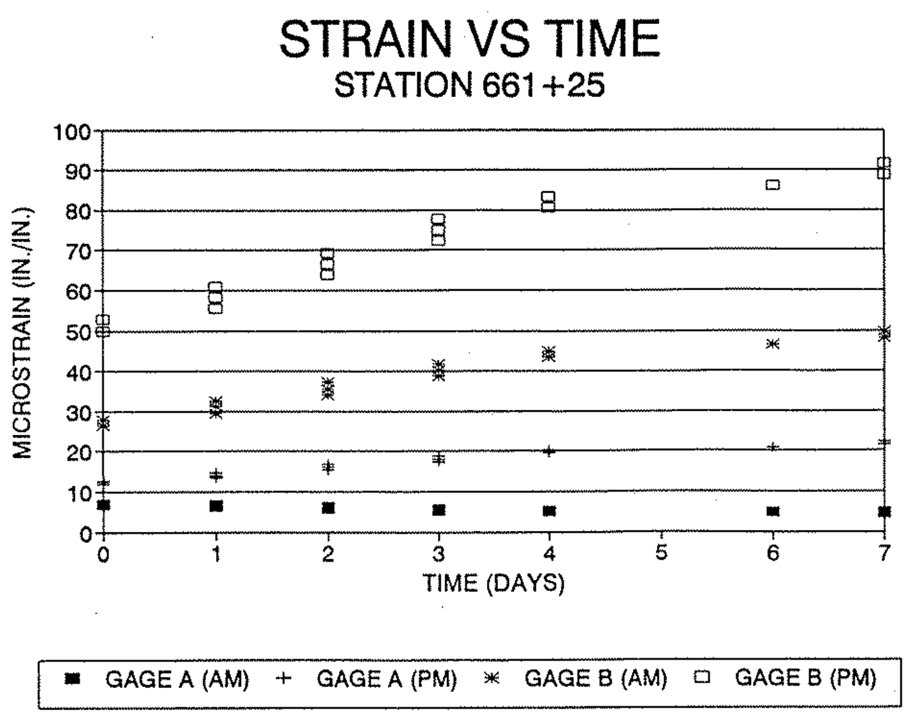
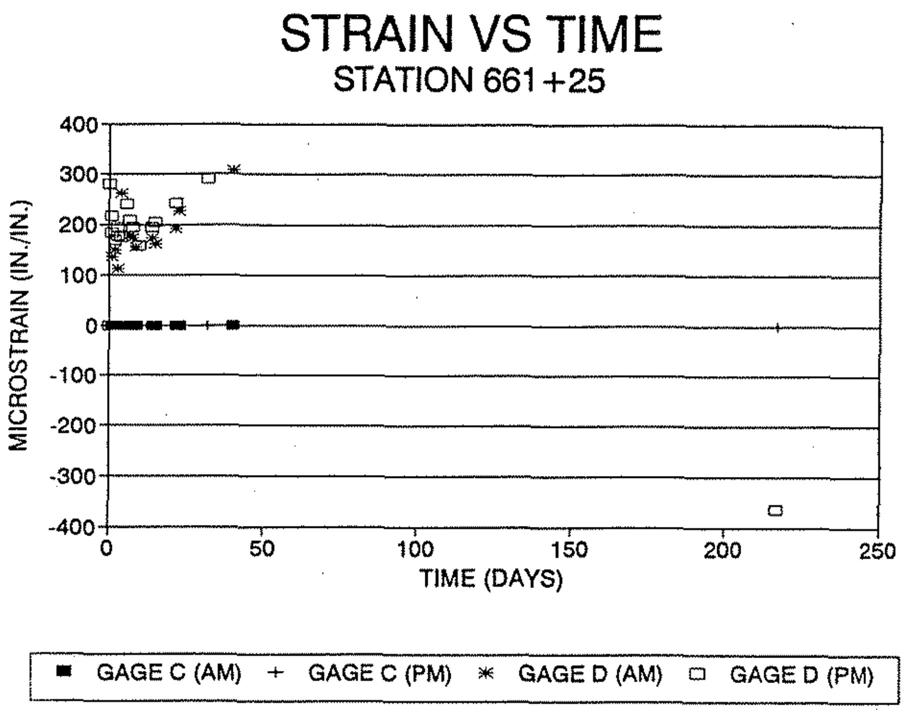




Figure B-3 Station 661+25, Gages A & B Strain



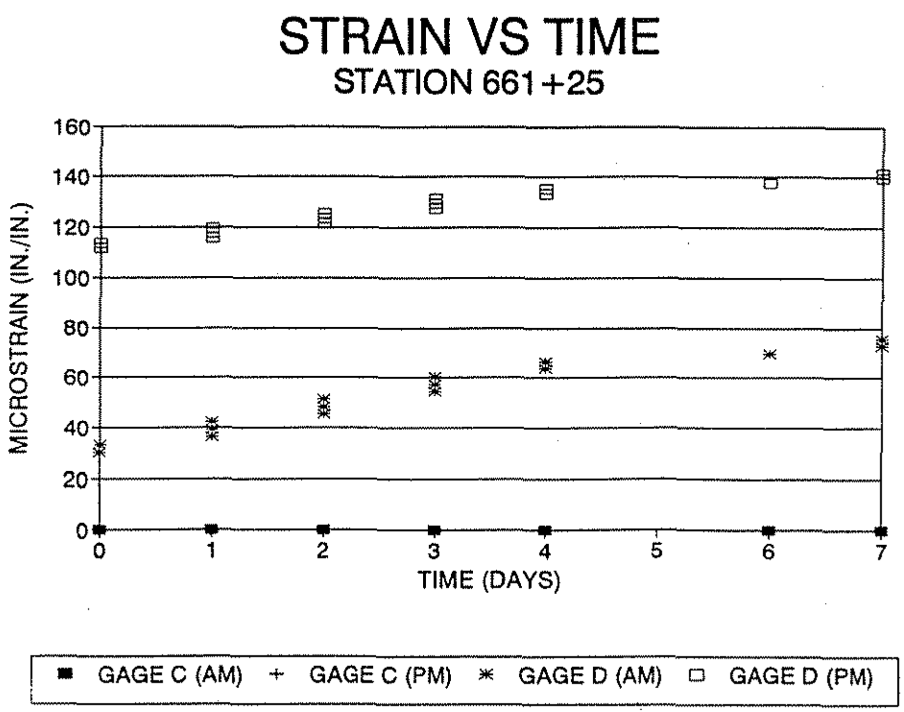
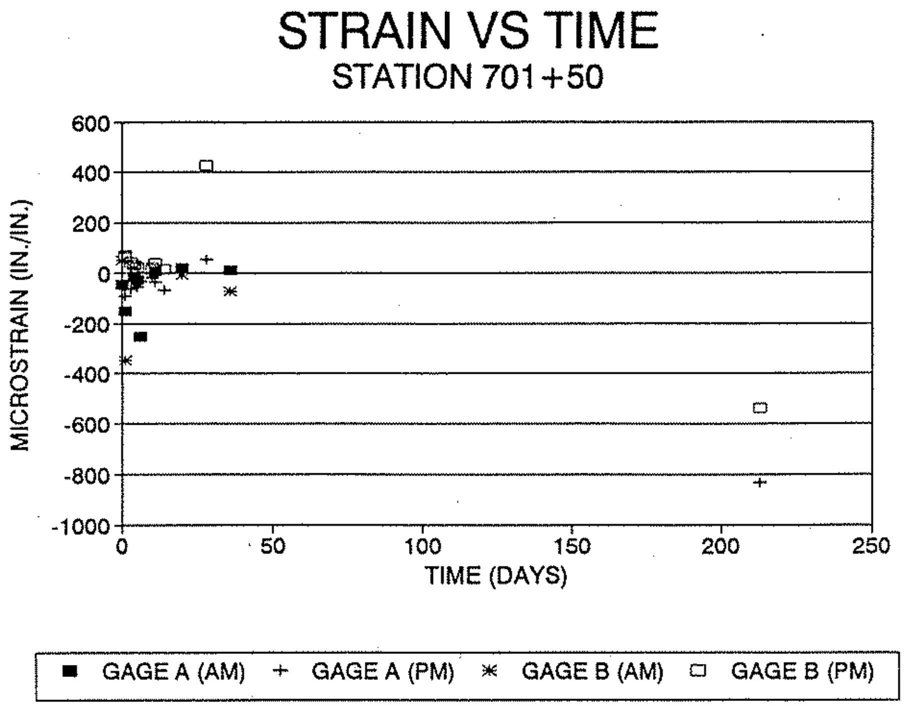




Figure B-4 Station 661+25, Gages C & D Strain



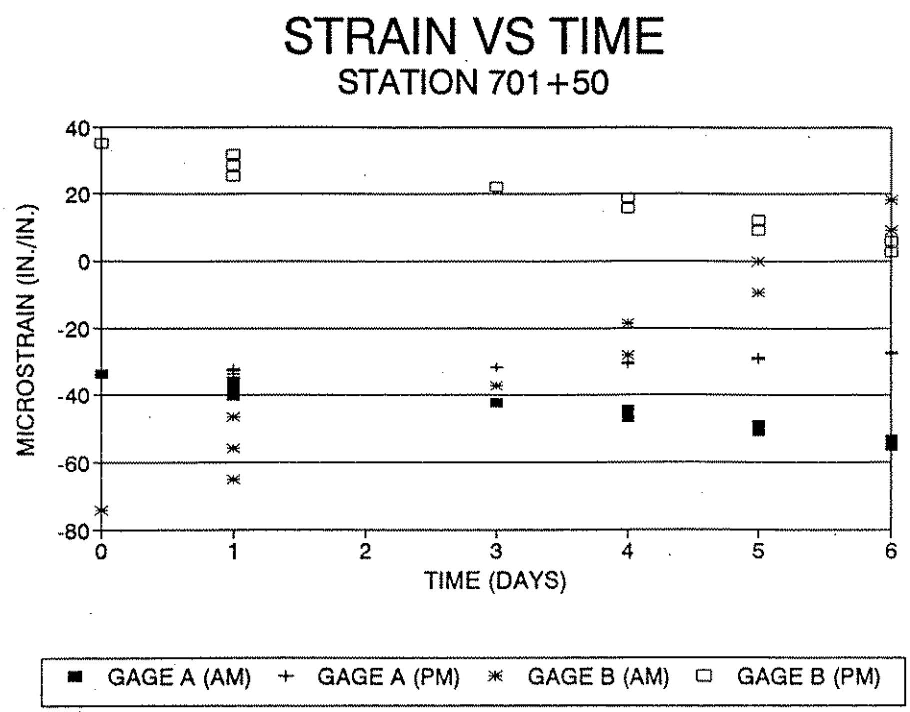
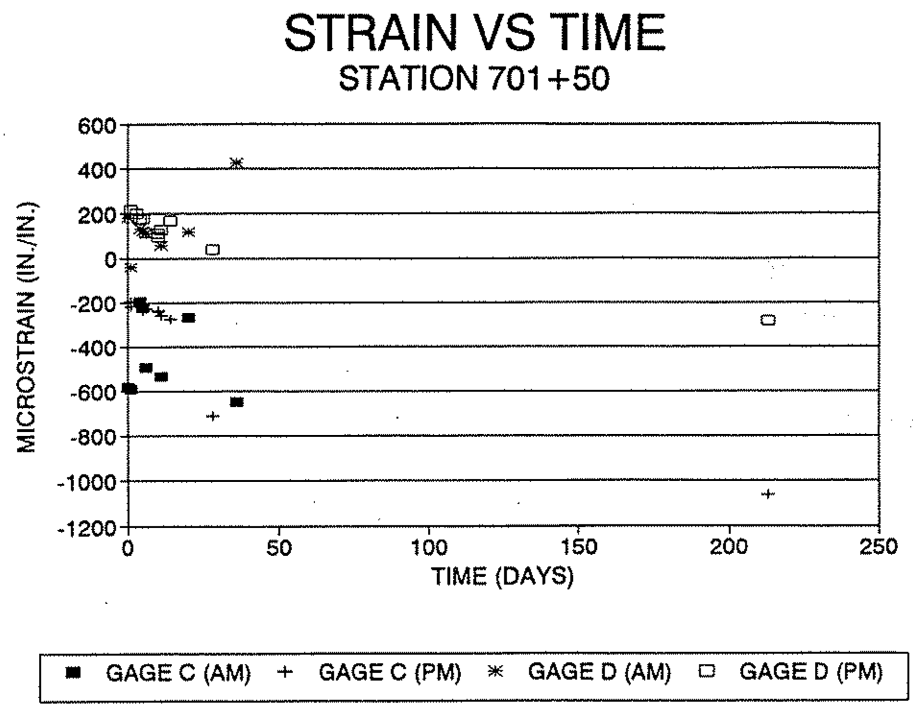




Figure B-5 Station 701+50, Gages A & B Strain



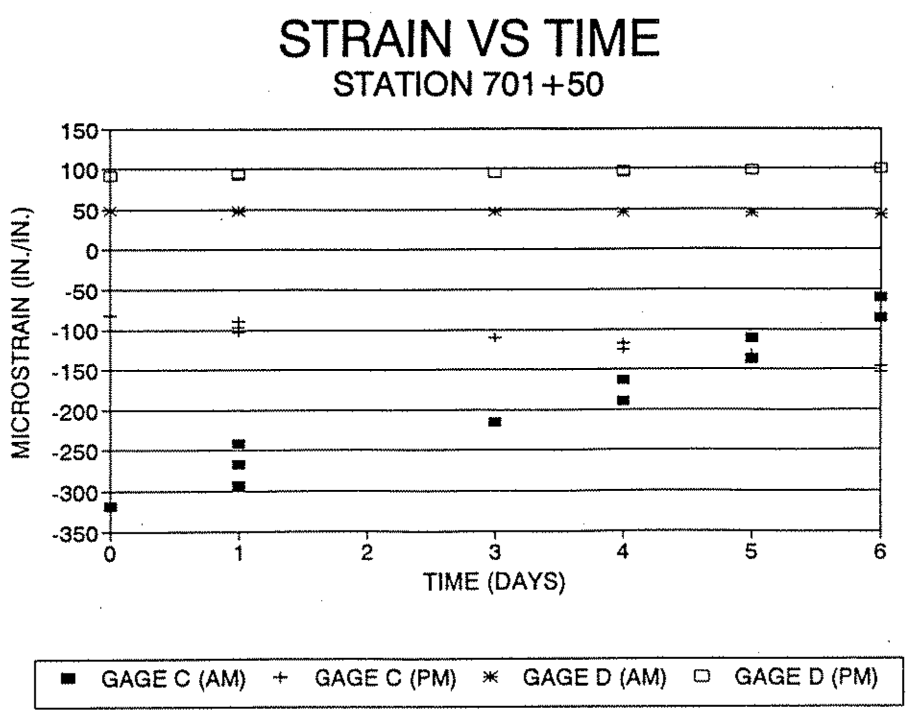
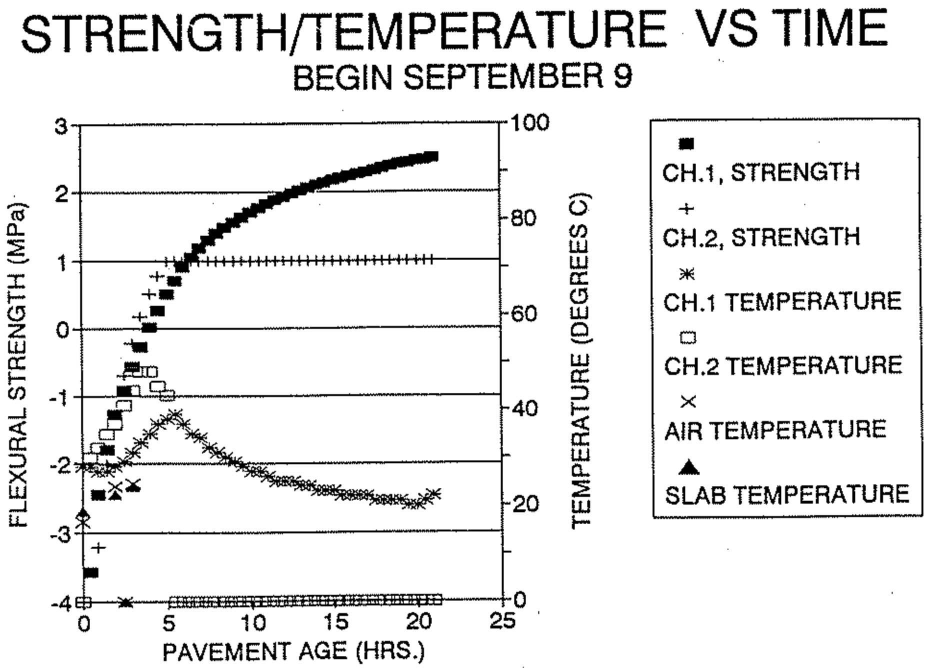




Figure B-6 Station 701+50, Gages C & D Strain

## APPENDIX C



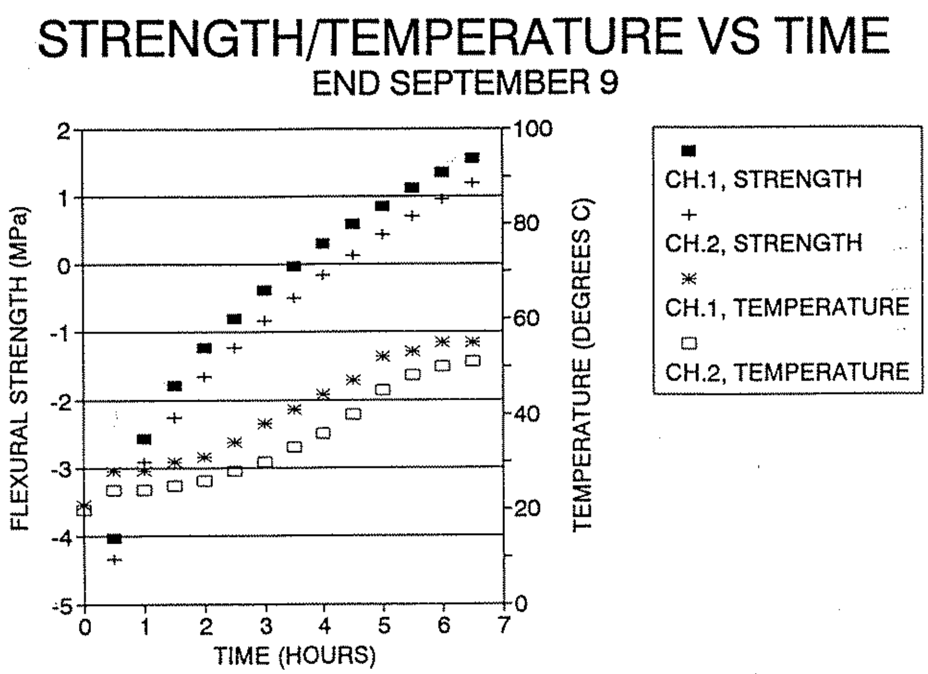
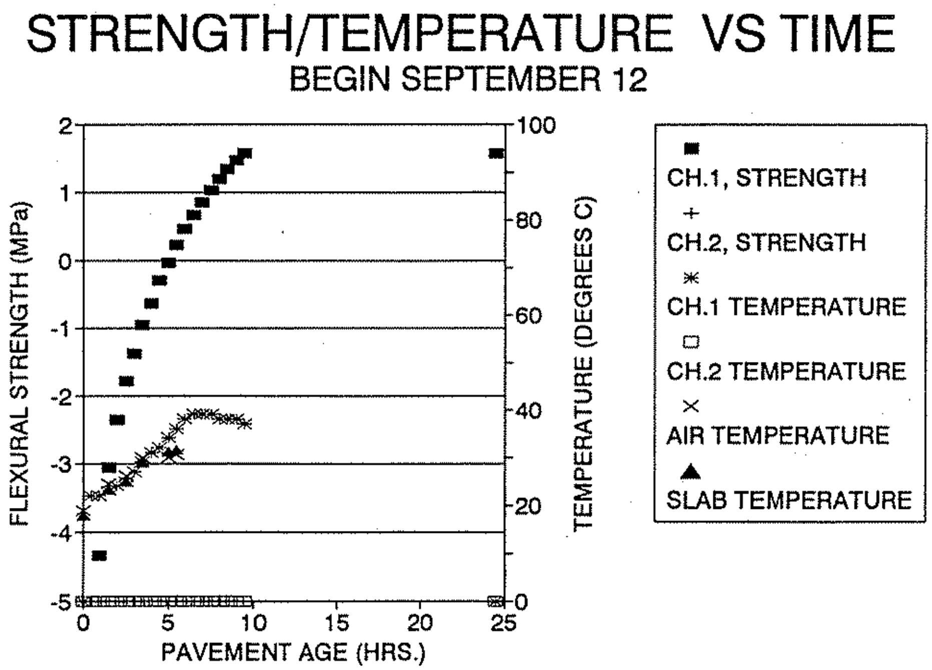




Figure C-1 Maturity, September 9, 1994



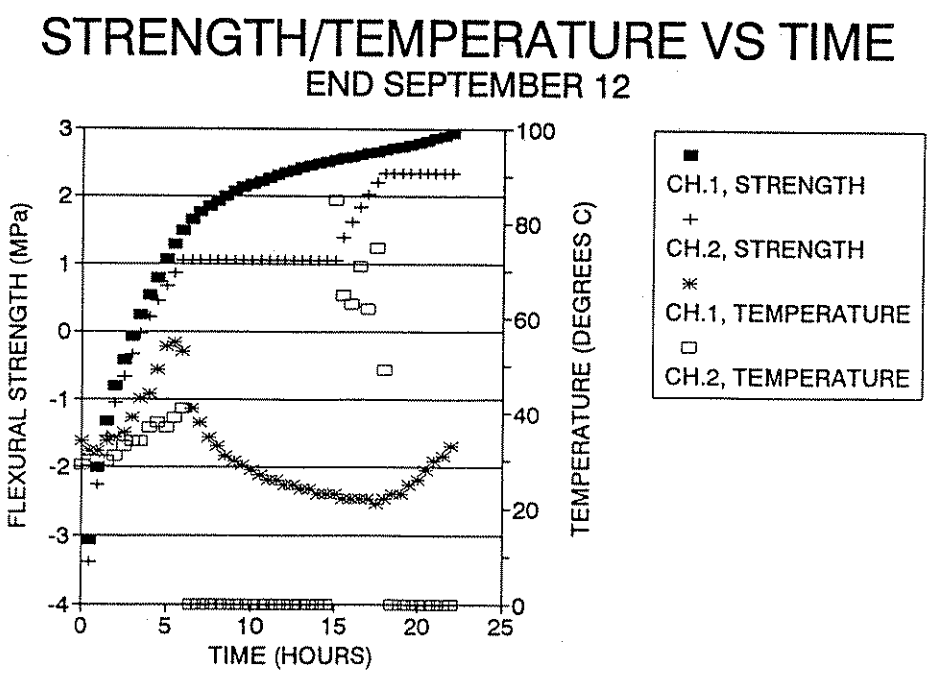
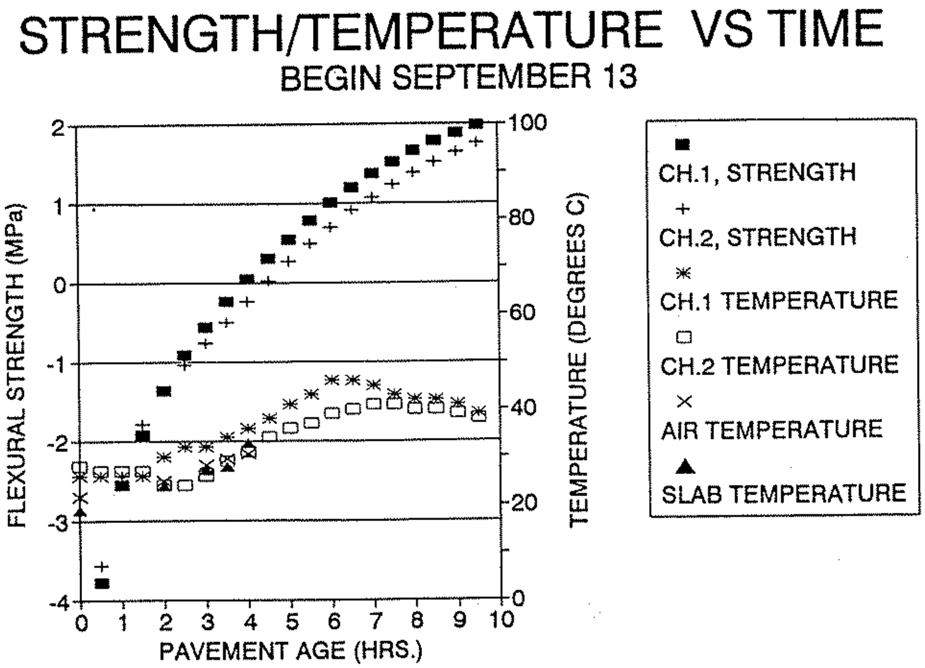




Figure C-2 Maturity, September 12, 1994



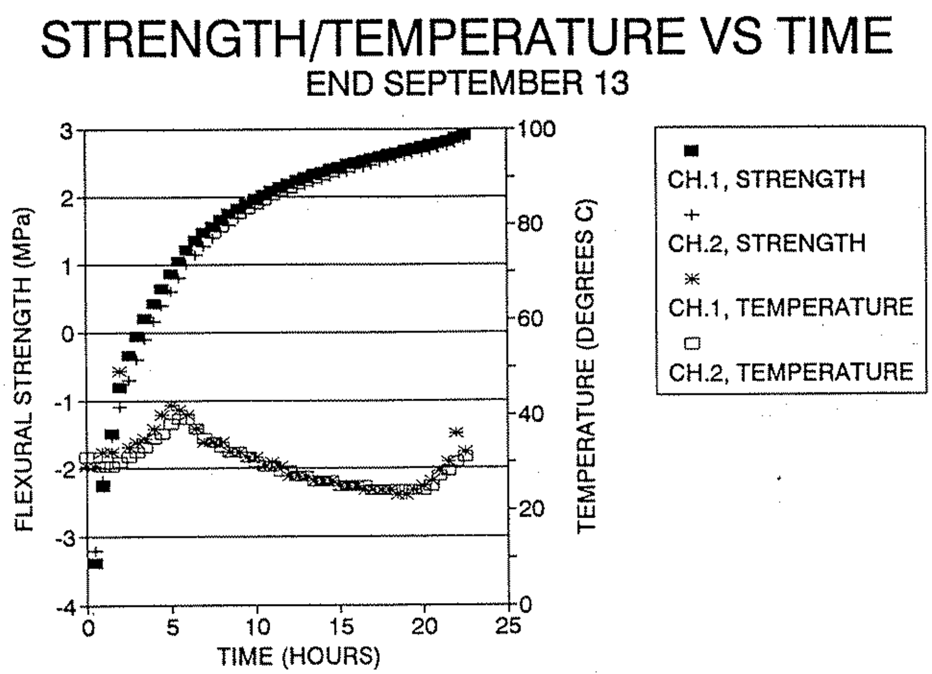
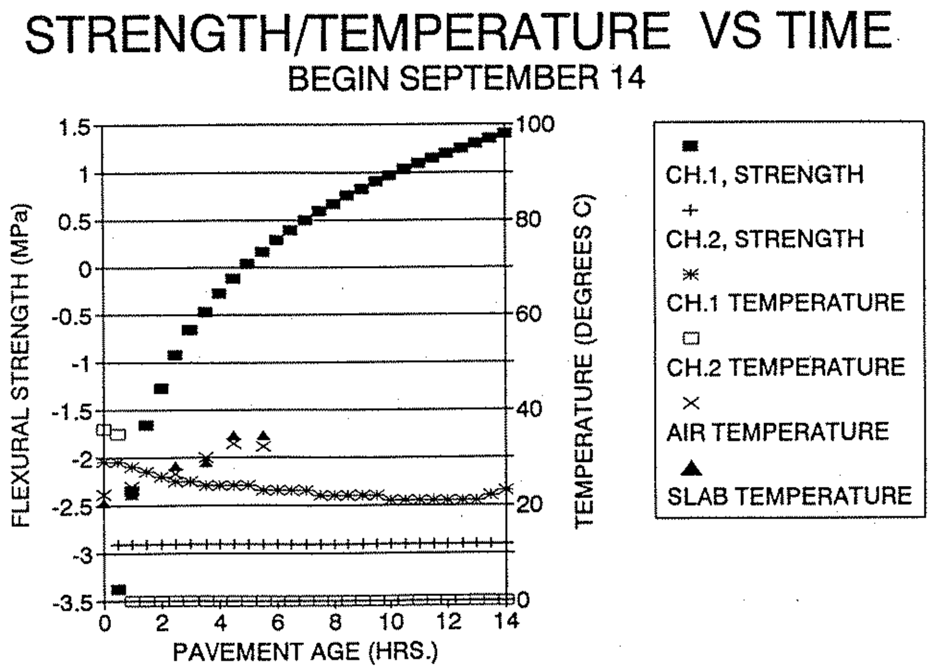




Figure C-3 Maturity, September 13, 1994



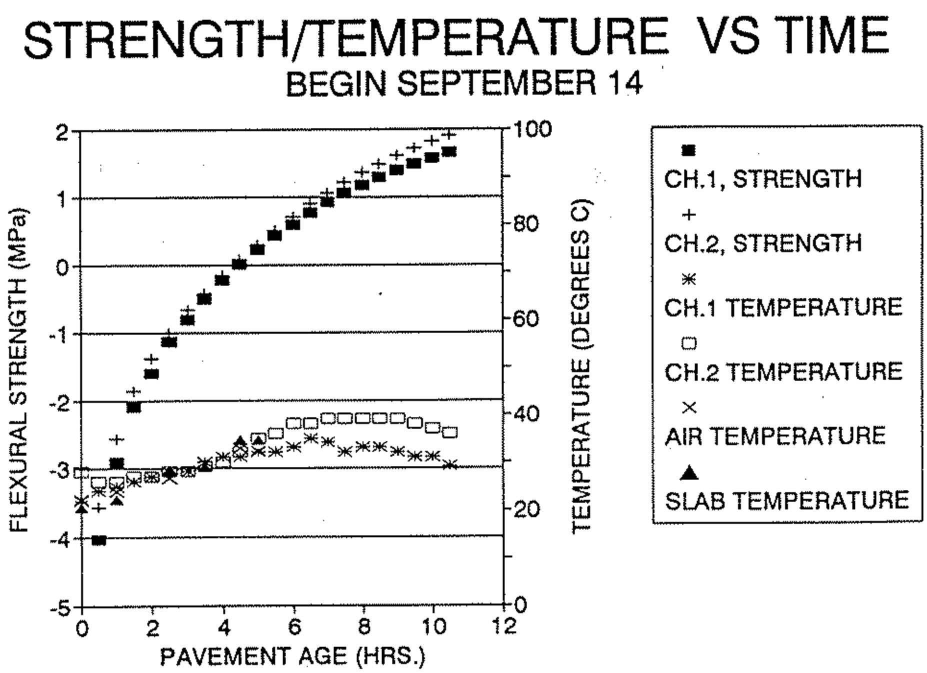




Figure C-4 Maturity, September 14, 1994

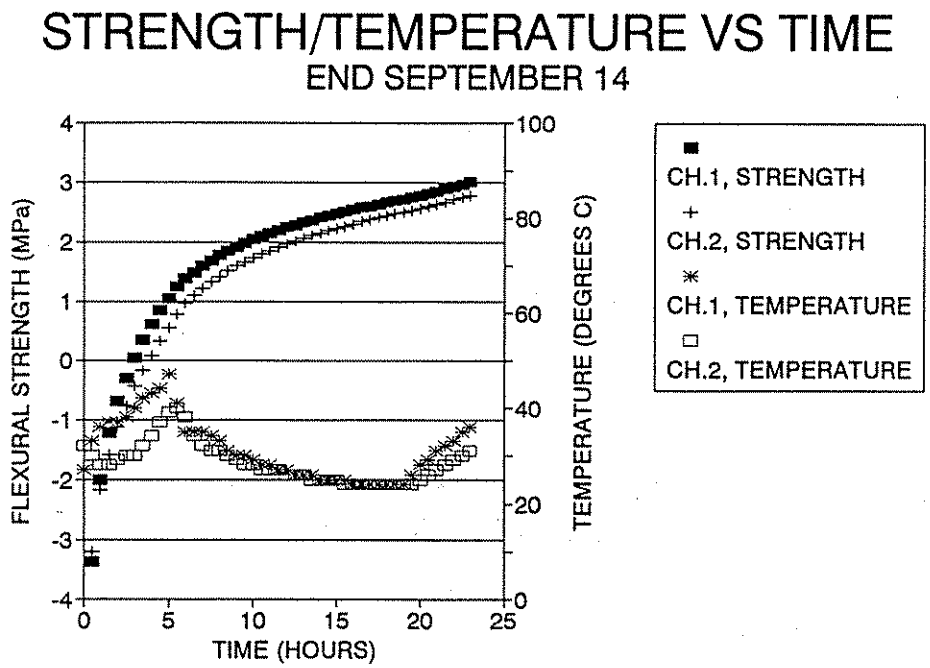
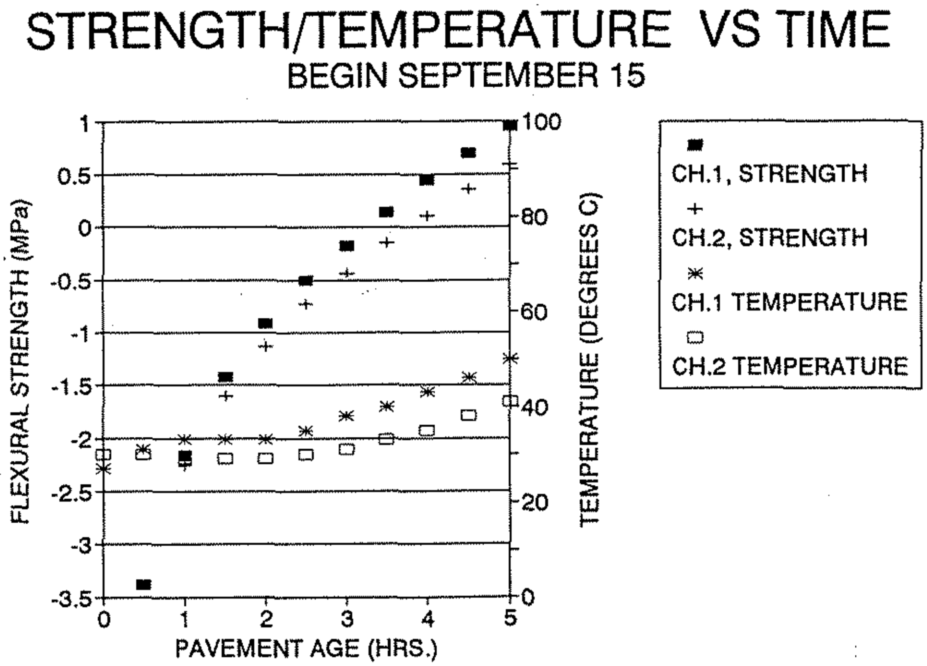



Figure C-5 Maturity, September 14, 1994 (Continued)



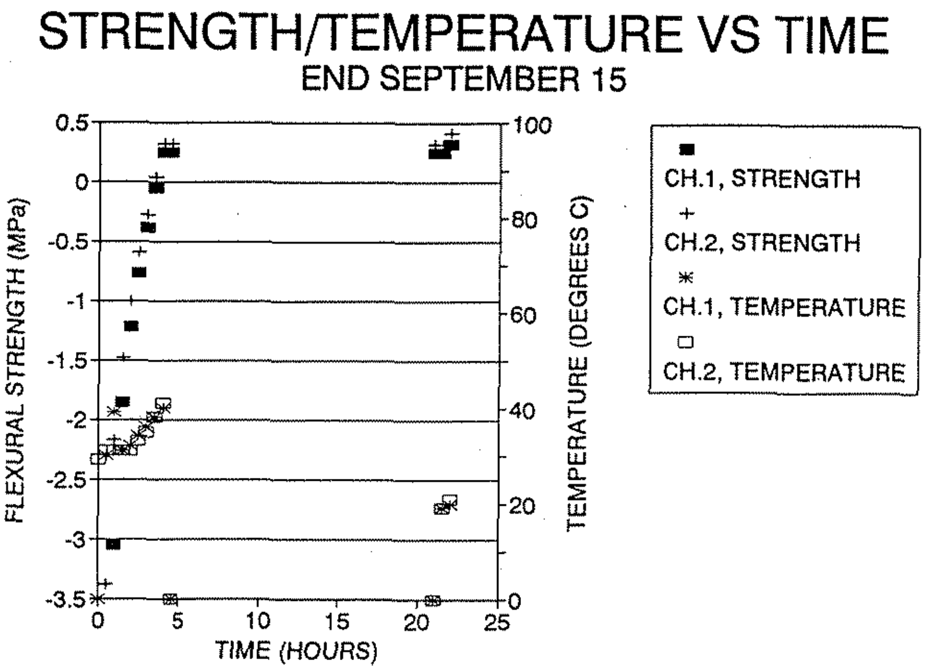
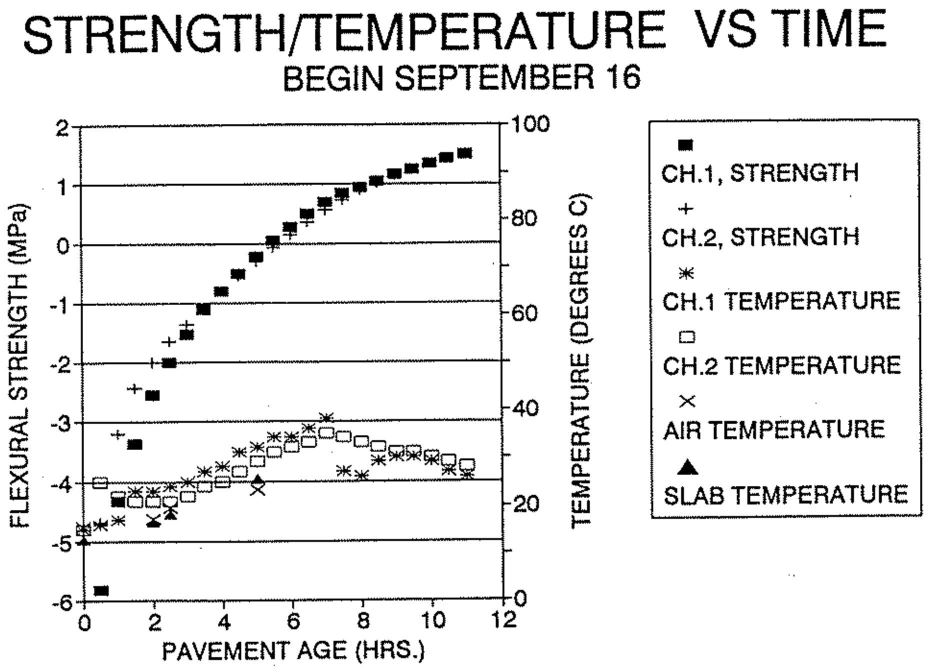




Figure C-6 Maturity, September 15, 1994



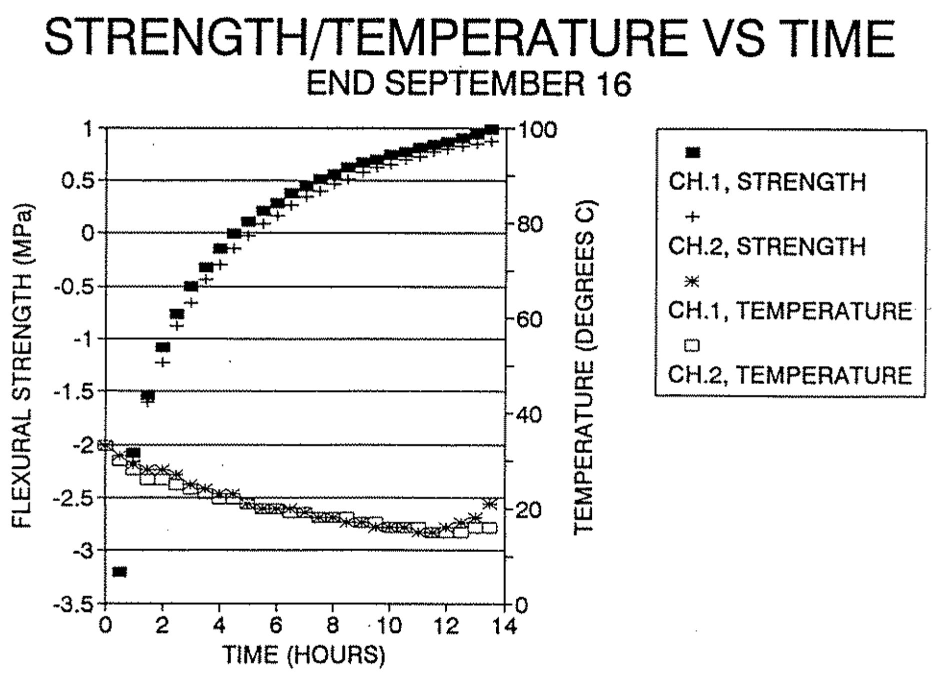




Figure C-7 Maturity, September 16, 1994

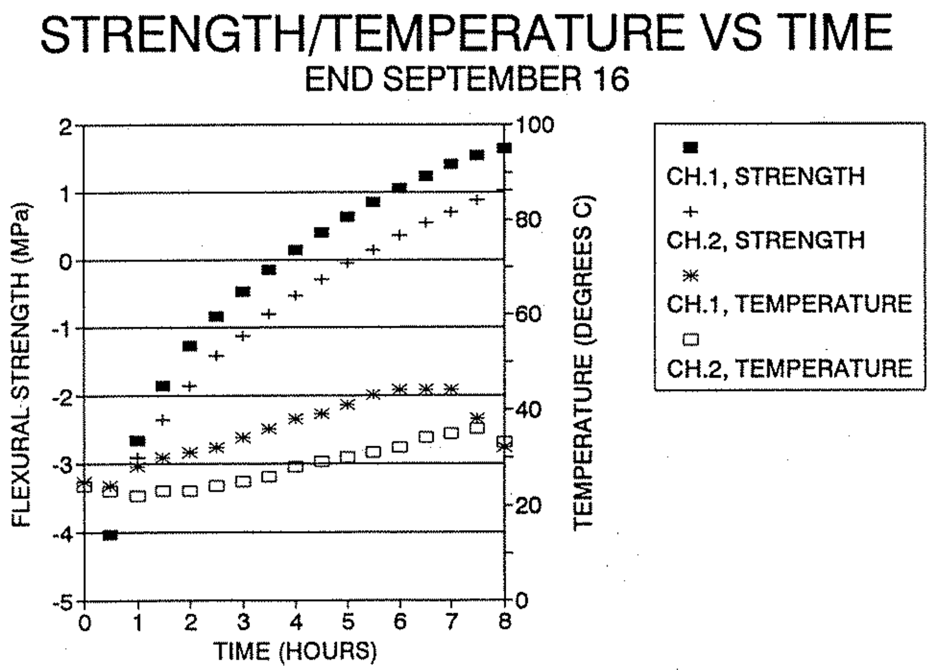
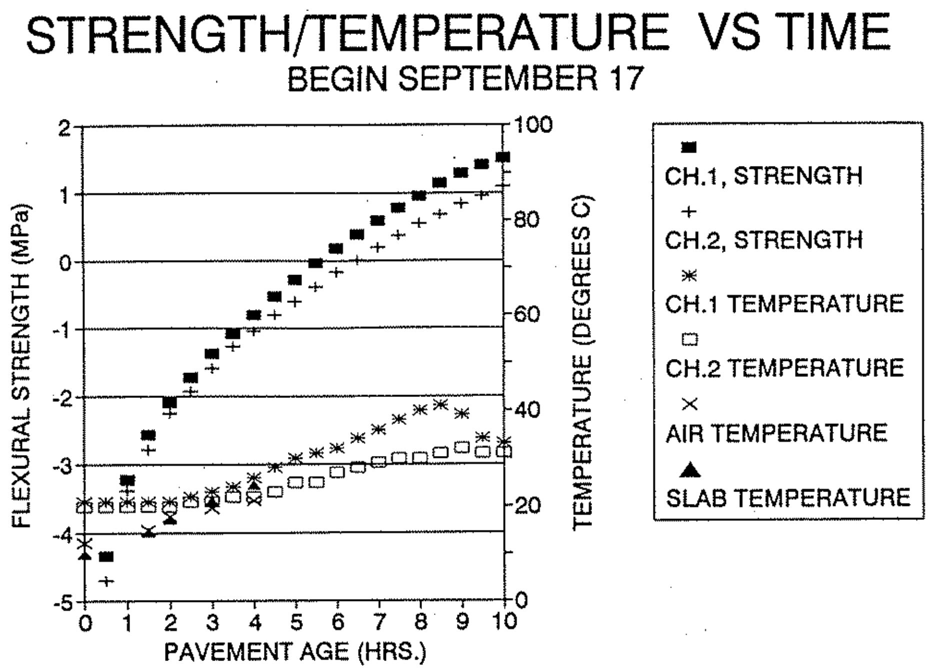



Figure C-8 Maturity, September 16, 1994 (Continued)



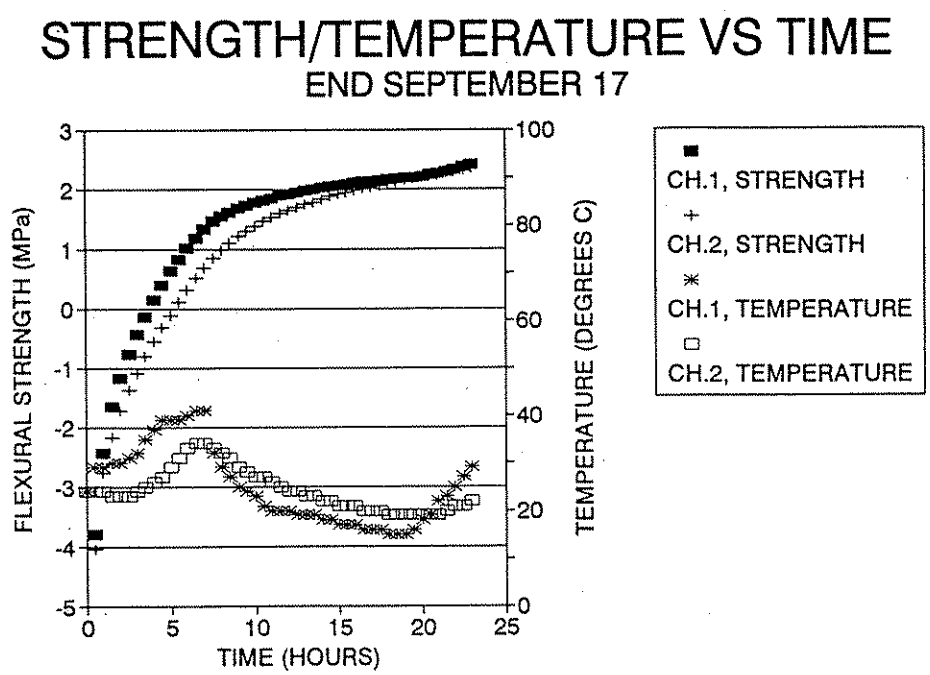
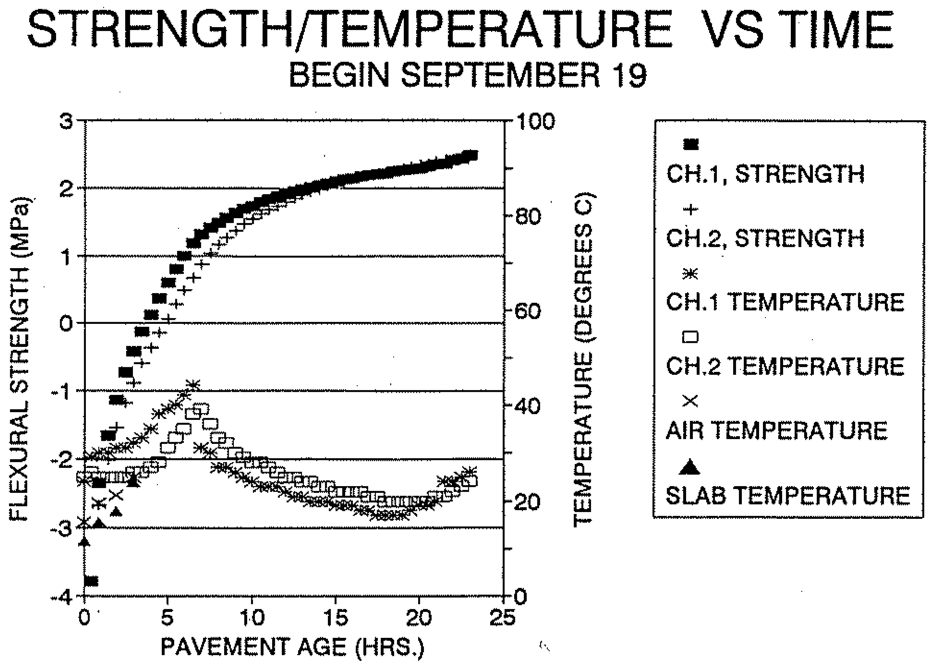




Figure C-9 Maturity, September 17, 1994



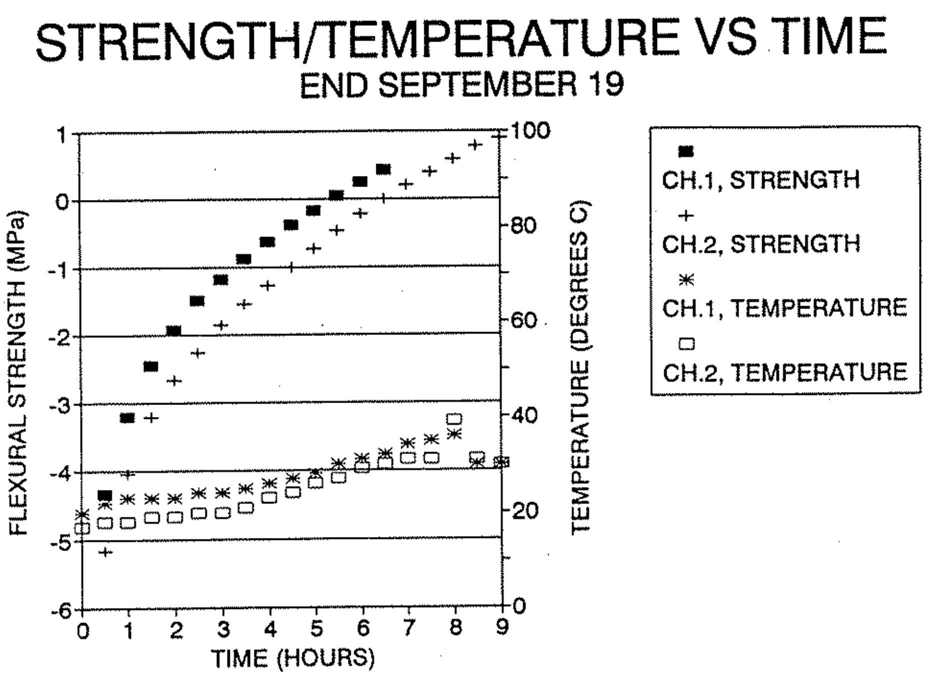
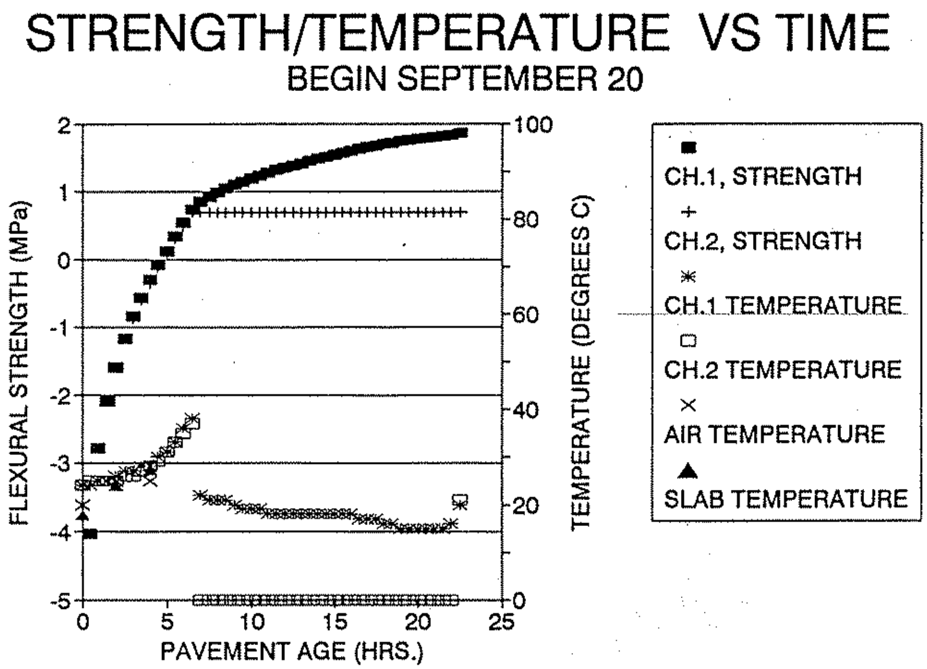




Figure C-10 Maturity, September 19, 1994



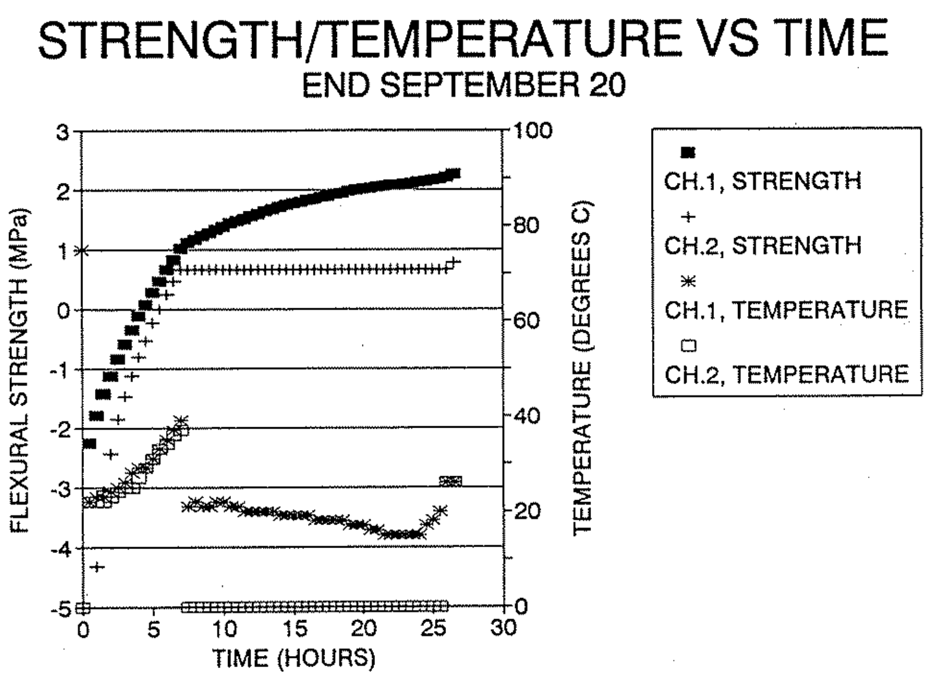
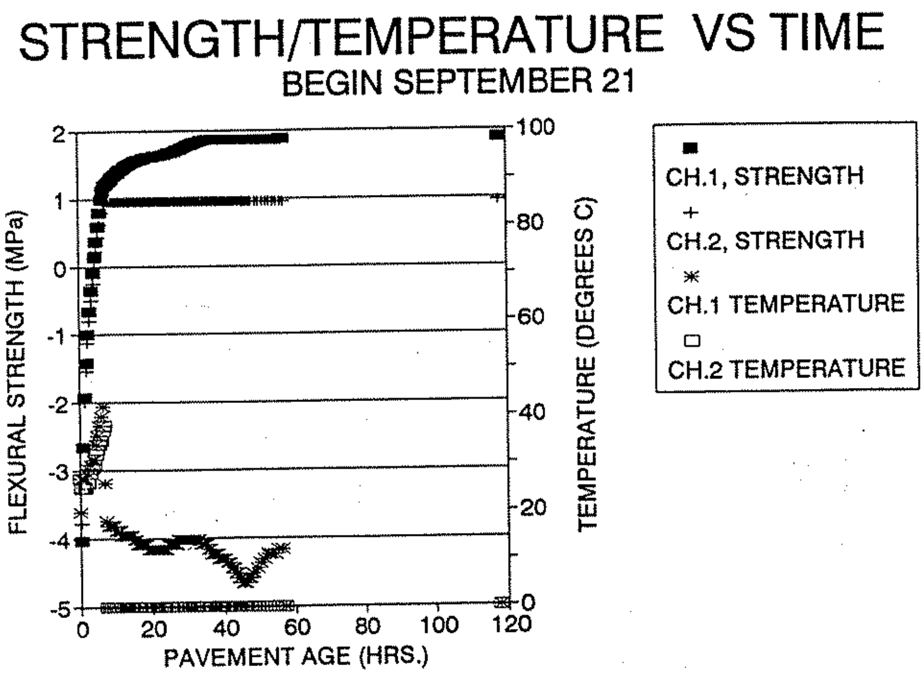




Figure C-11 Maturity, September 20, 1994



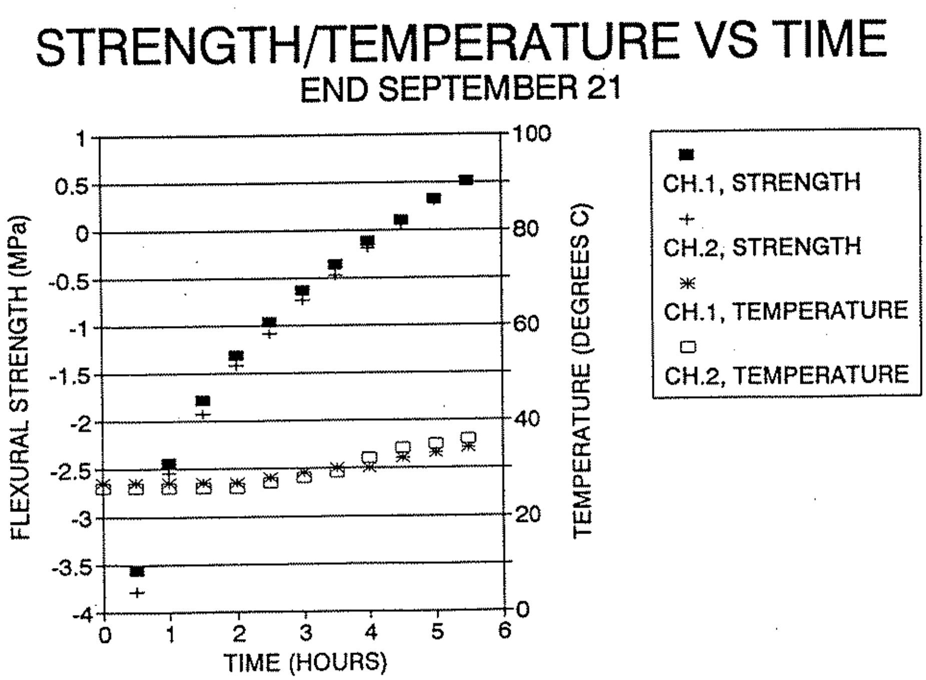
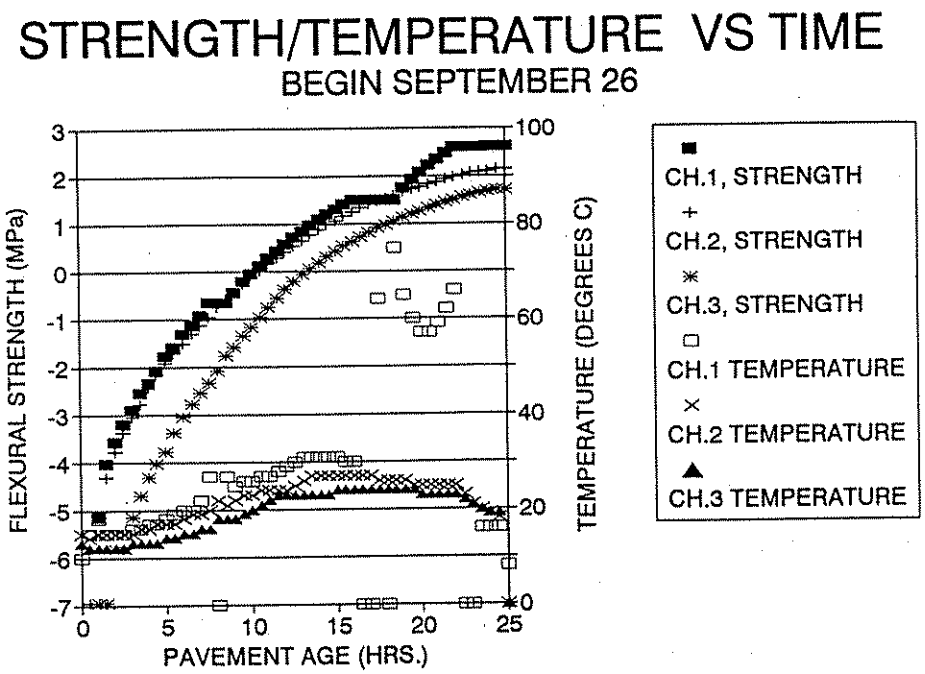




Figure C-12 Maturity, September 21, 1994



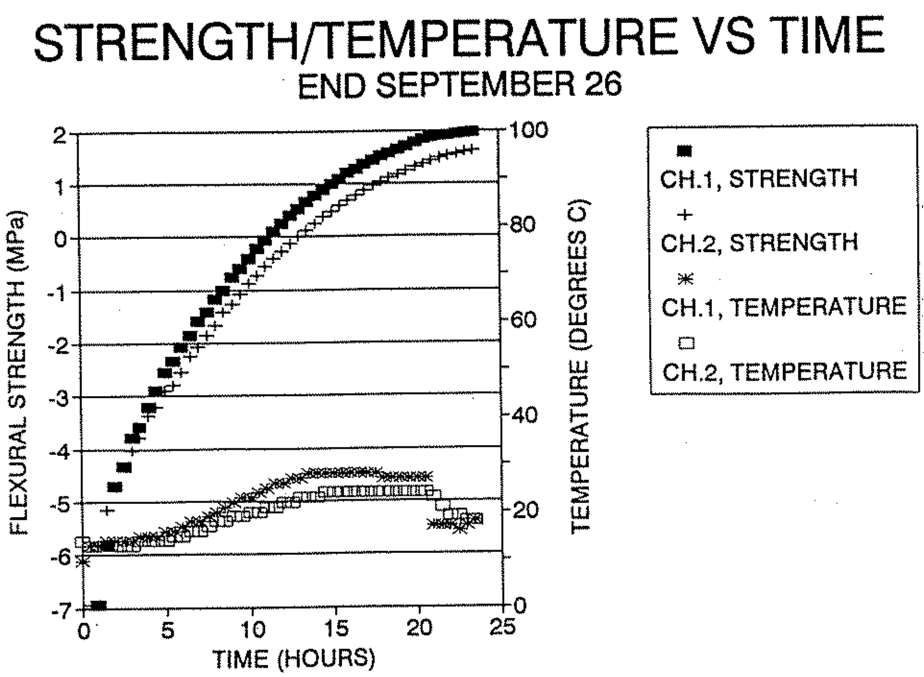
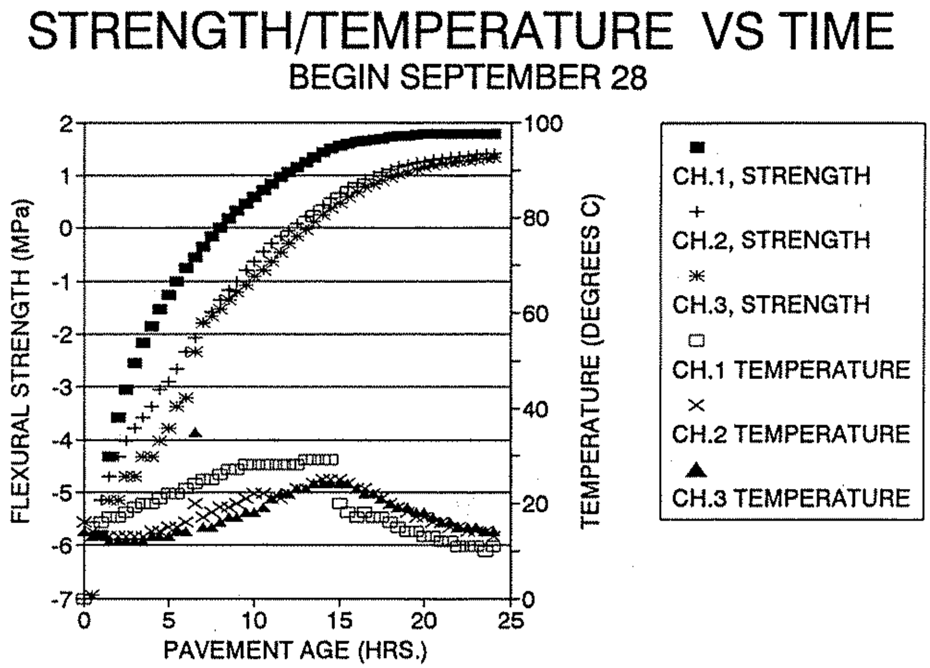




Figure C-13 Maturity, September 26, 1994



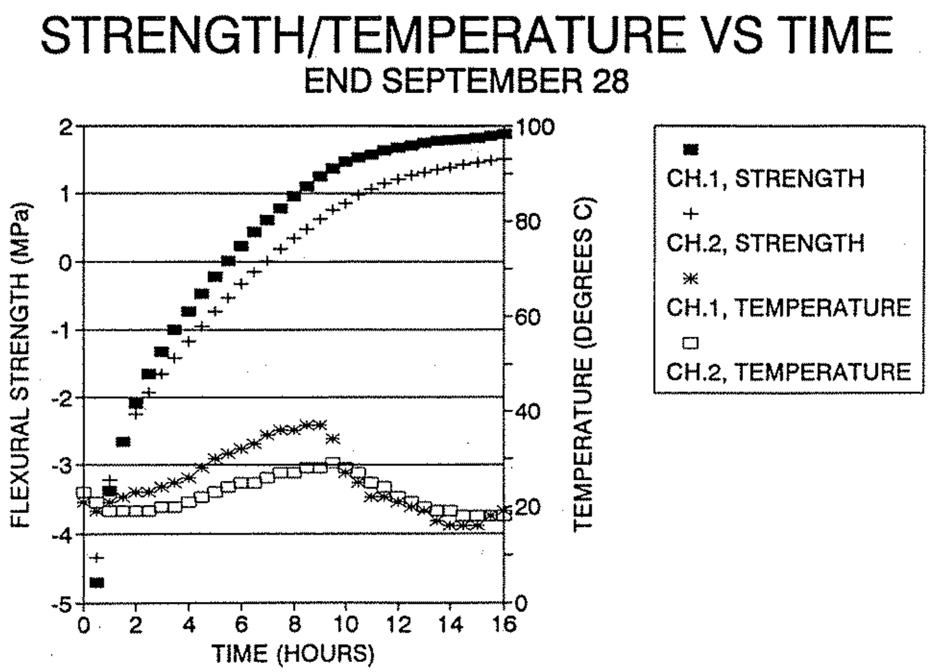




Figure C-14 Maturity, September 28, 1994

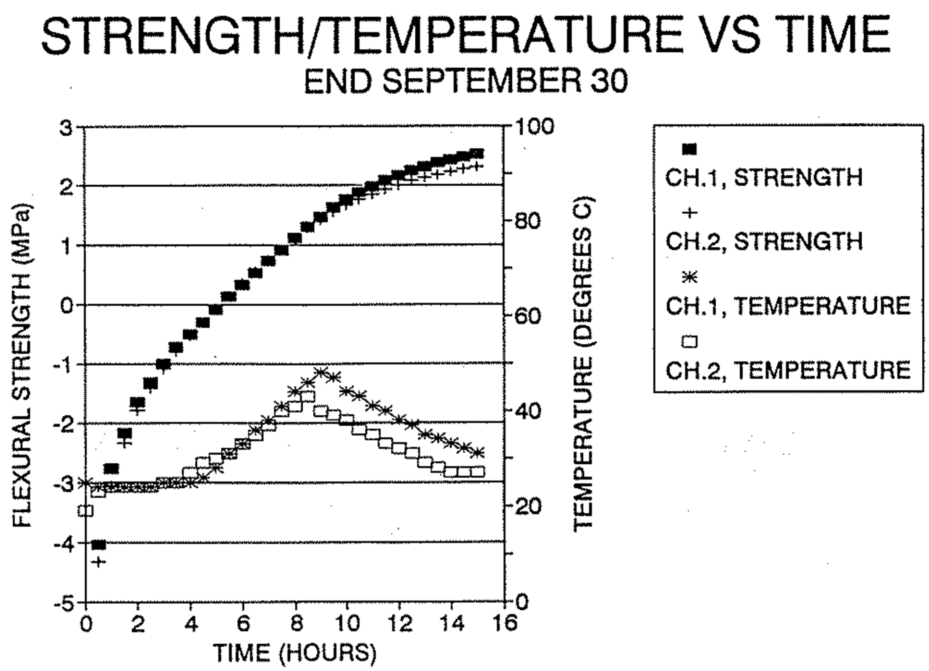
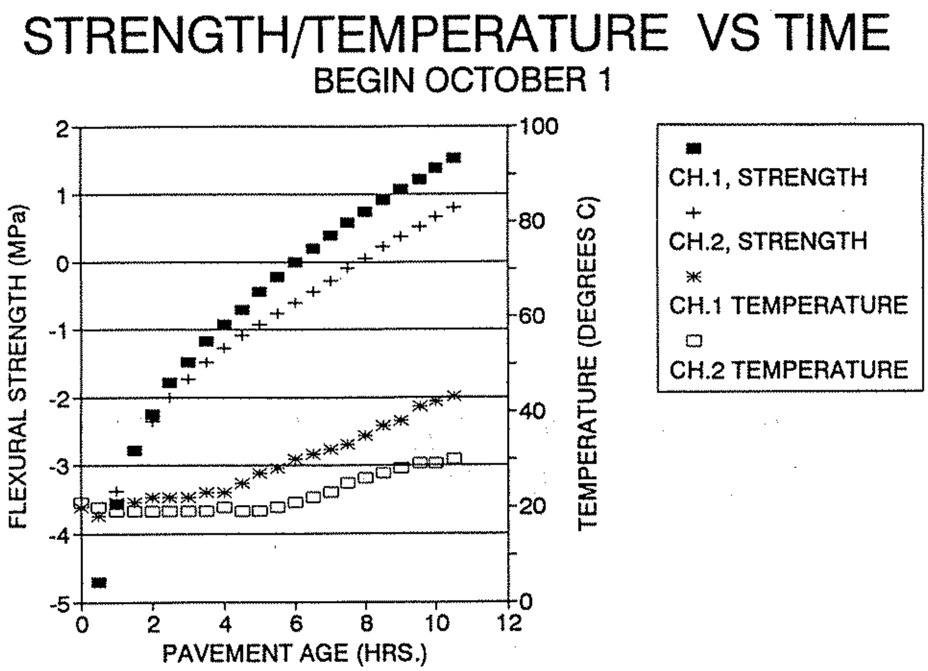



Figure C-15 Maturity, September 30, 1994



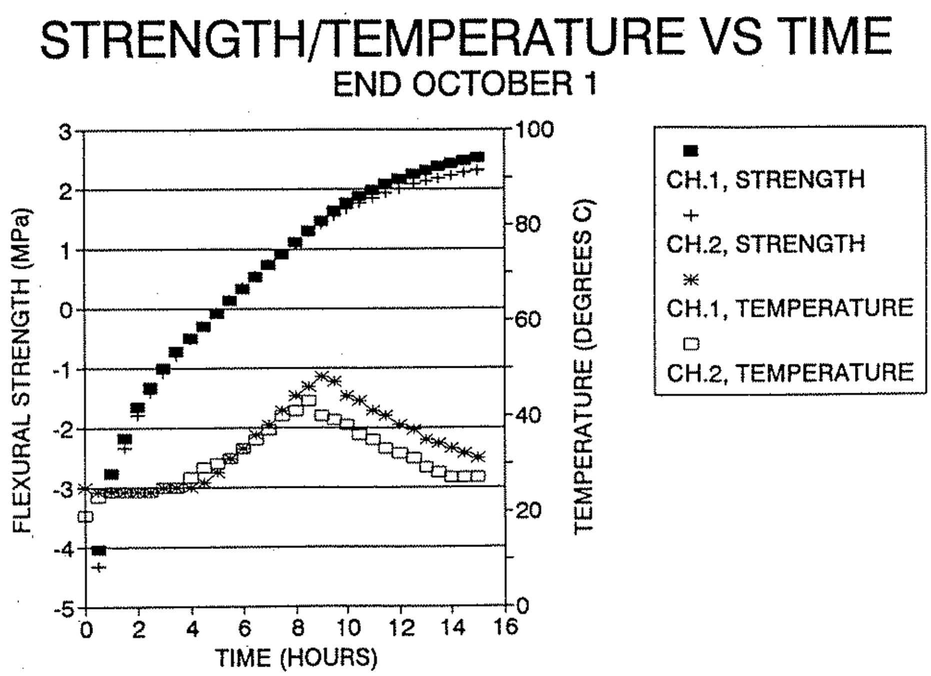
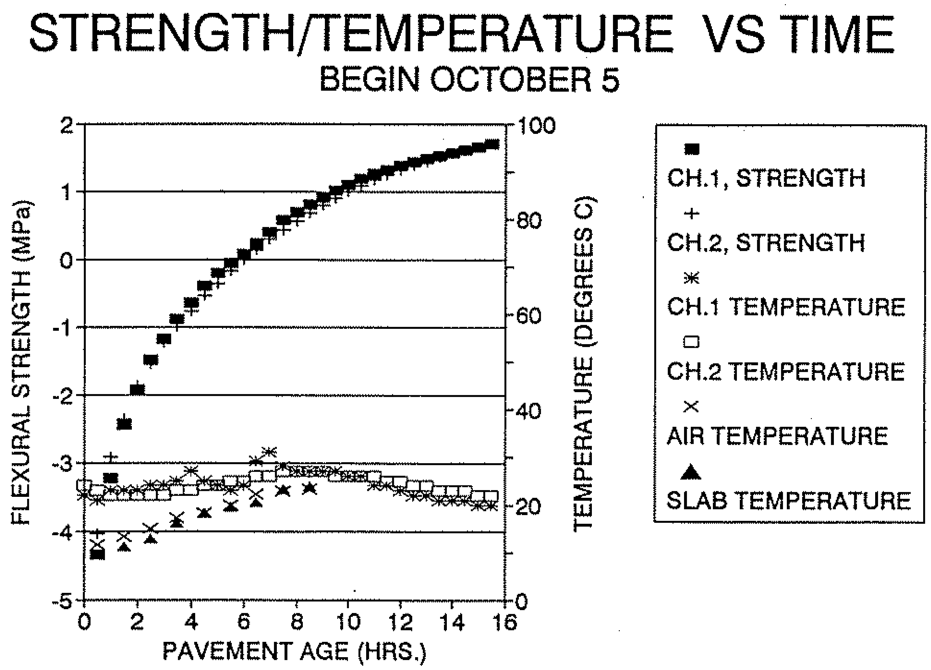




Figure C-16 Maturity, October 1, 1994



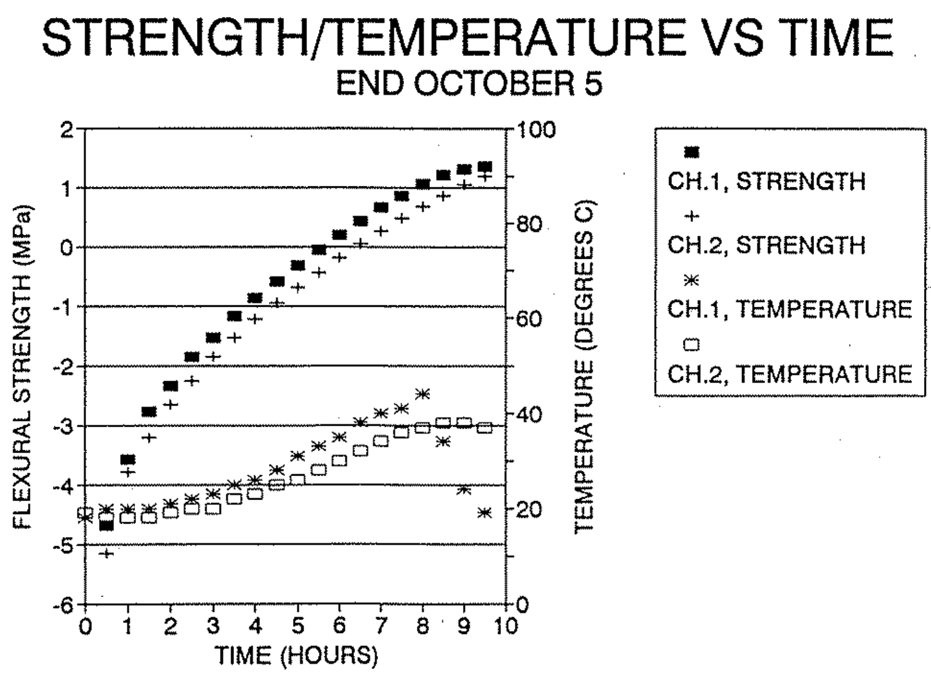
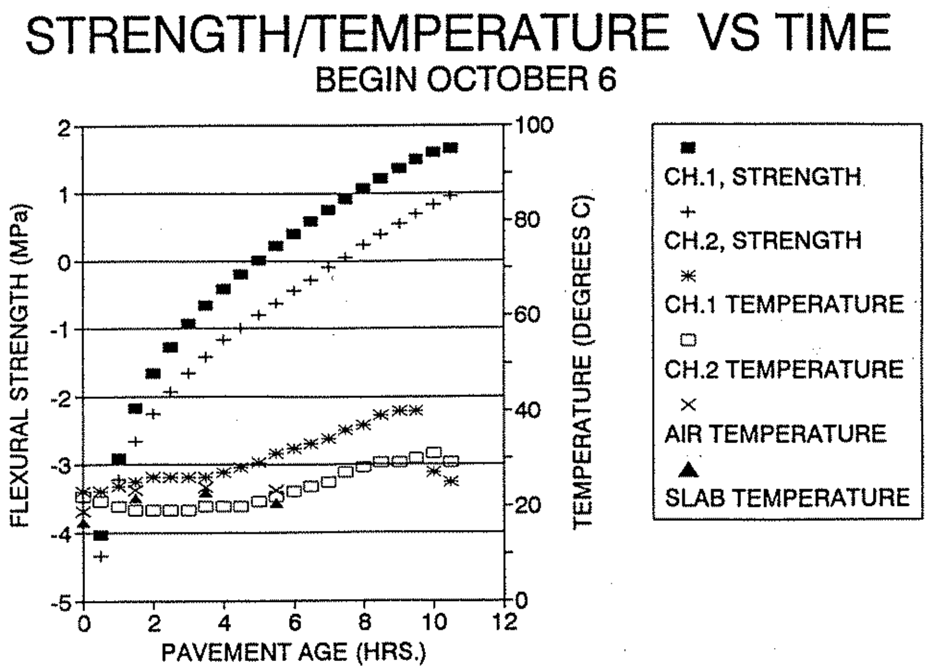




Figure C-17 Maturity, October 5, 1994



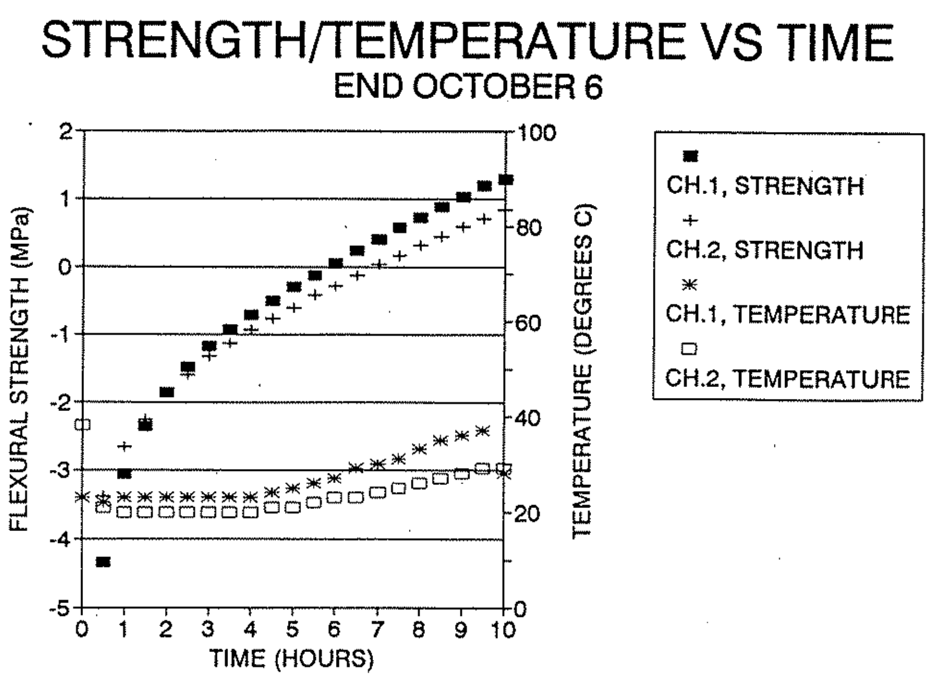
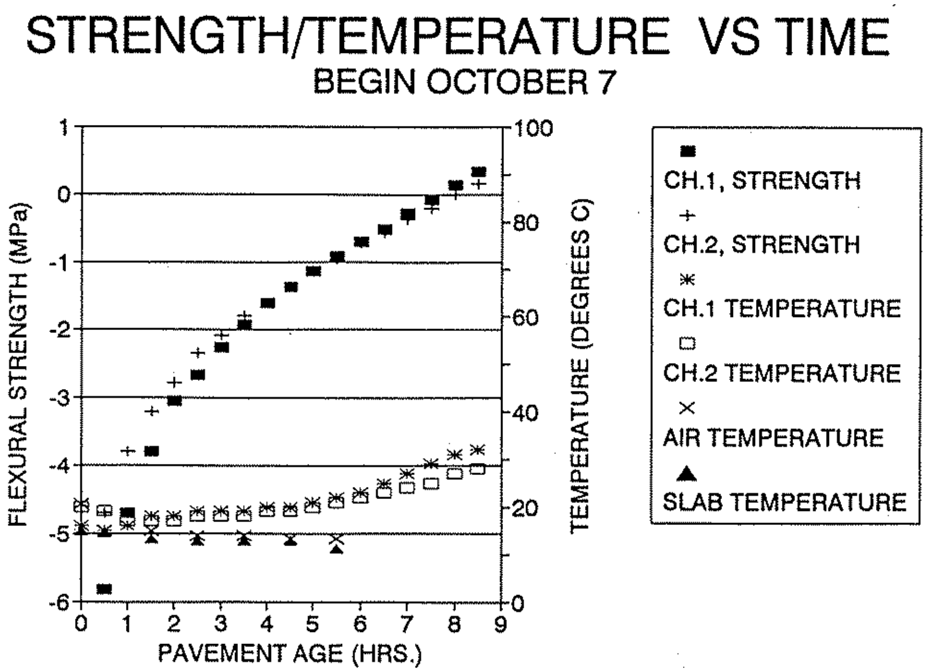




Figure C-18 Maturity, October 6, 1994



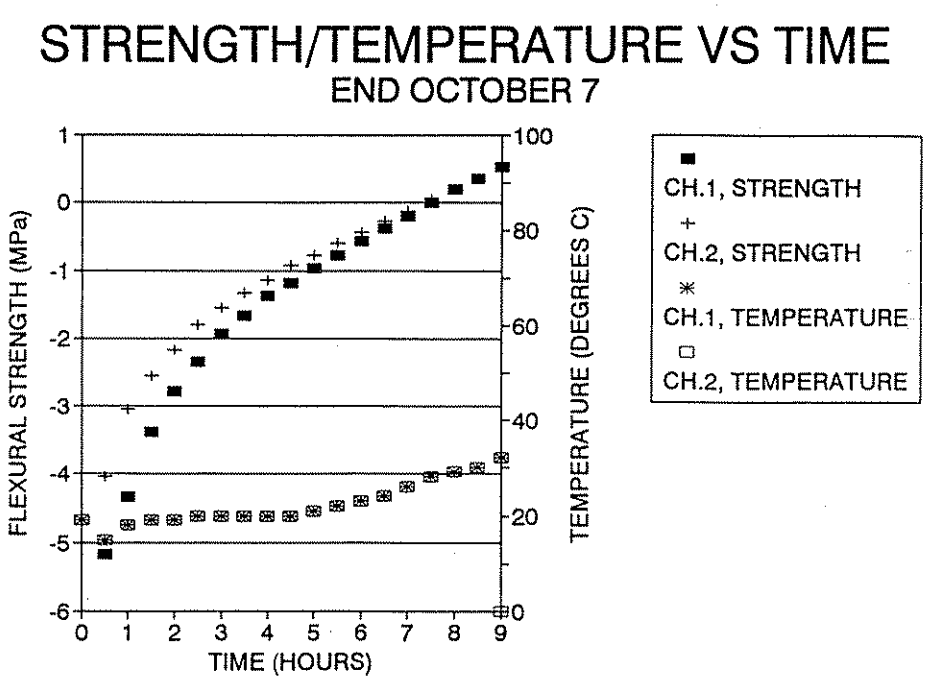
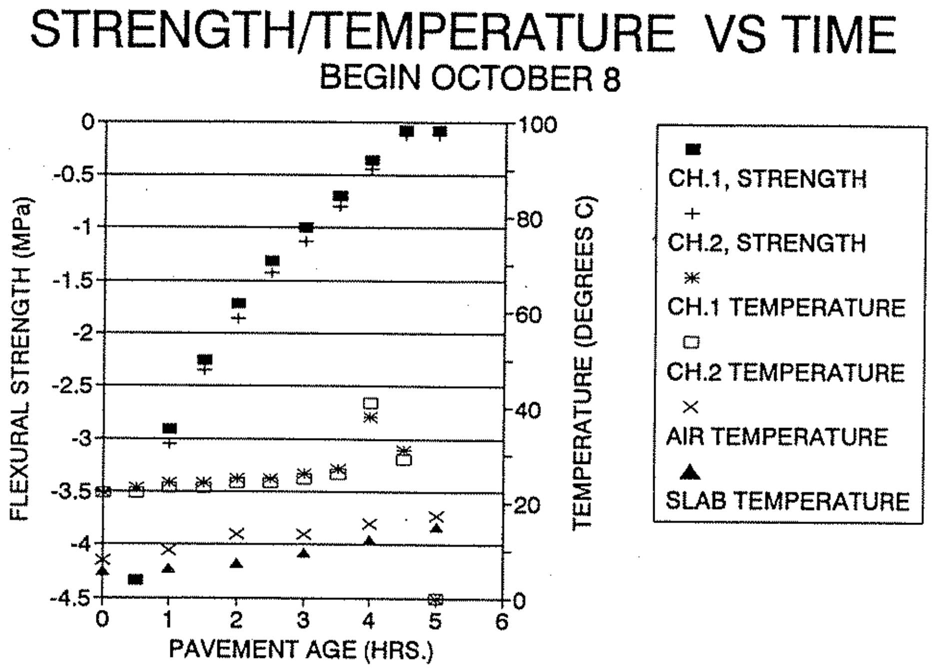




Figure C-19 Maturity, October 7, 1994



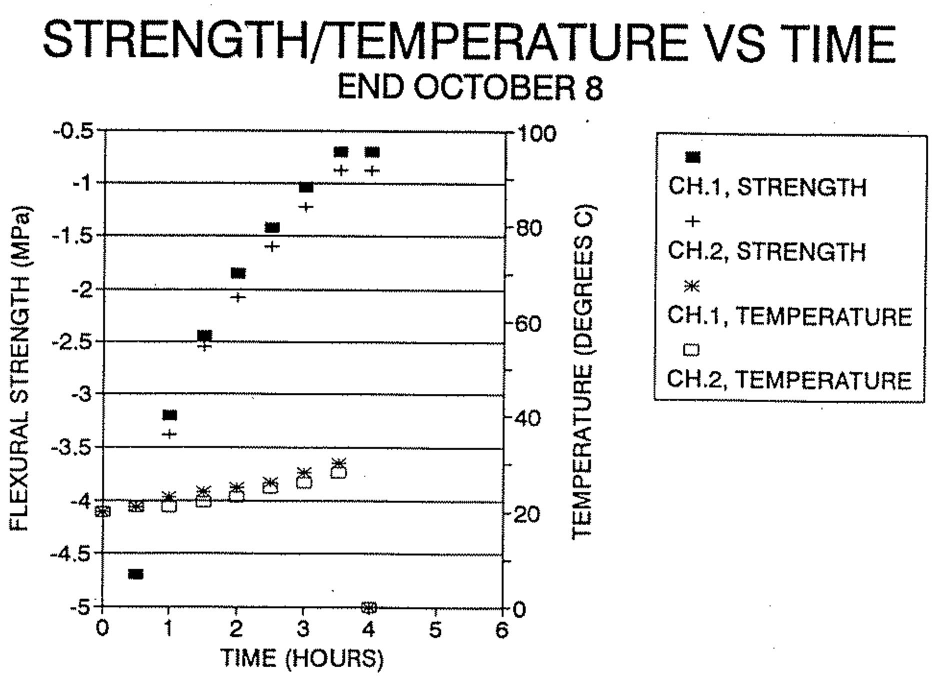




Figure C-20 Maturity, October 8, 1994

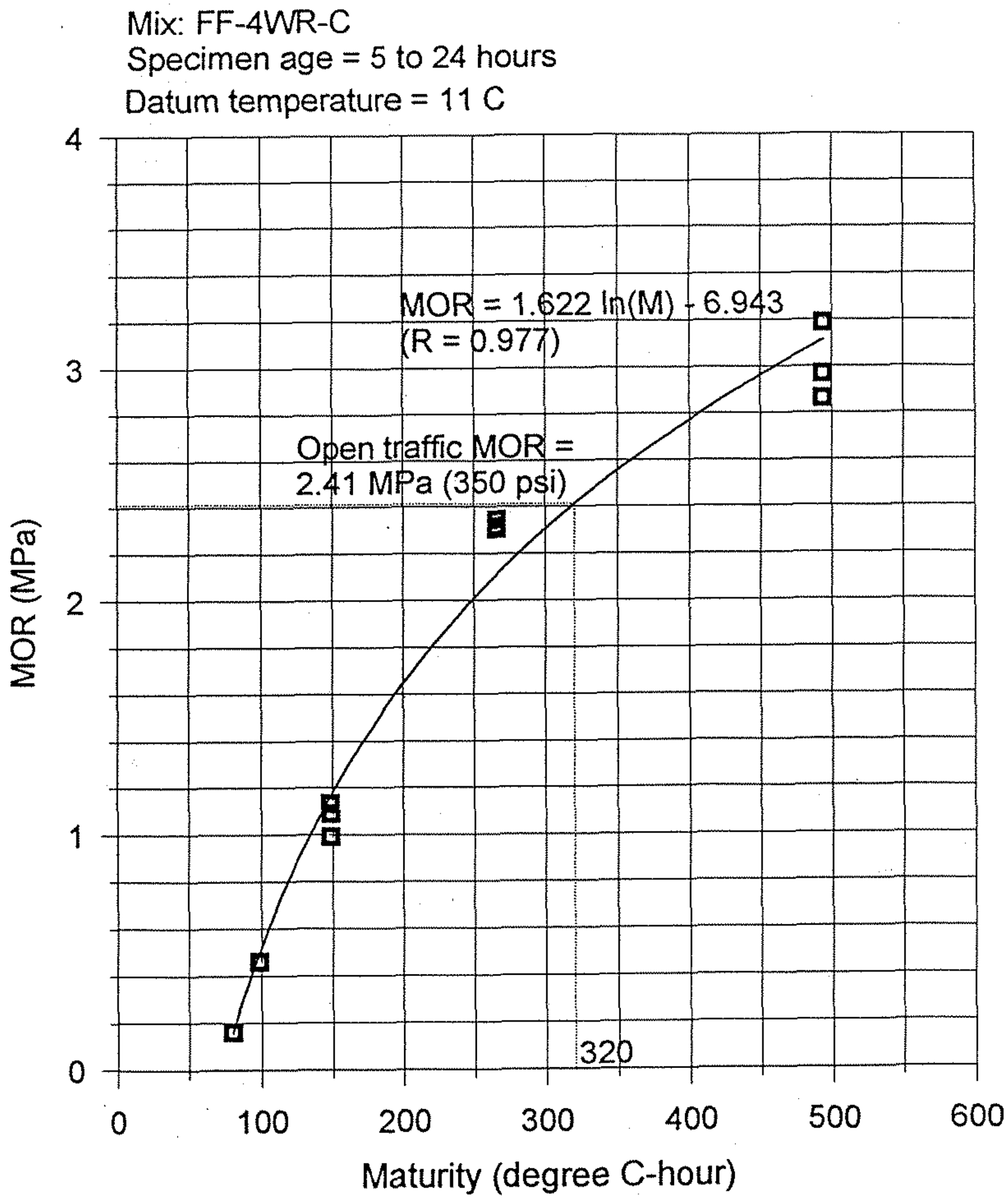
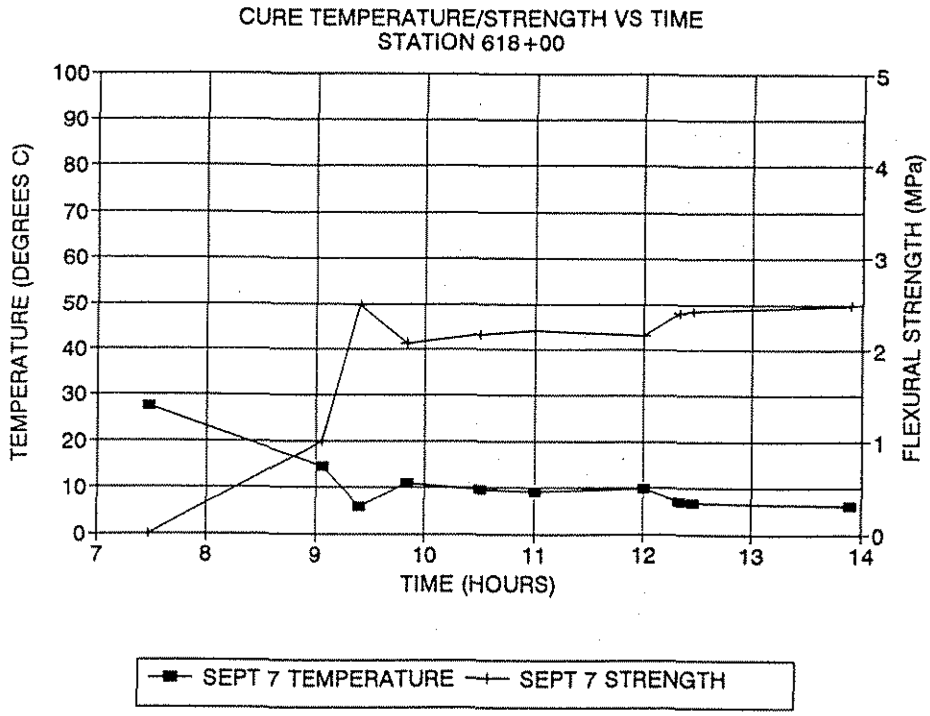



Figure C-21 Relationship Between Maturity and MOR (Strength)

**APPENDIX D**



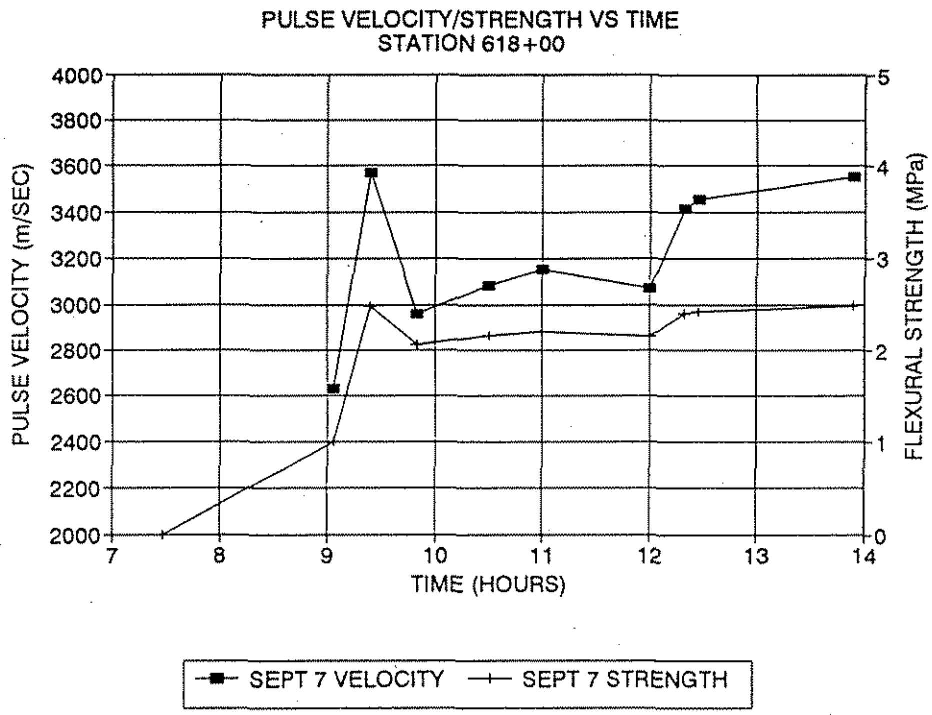
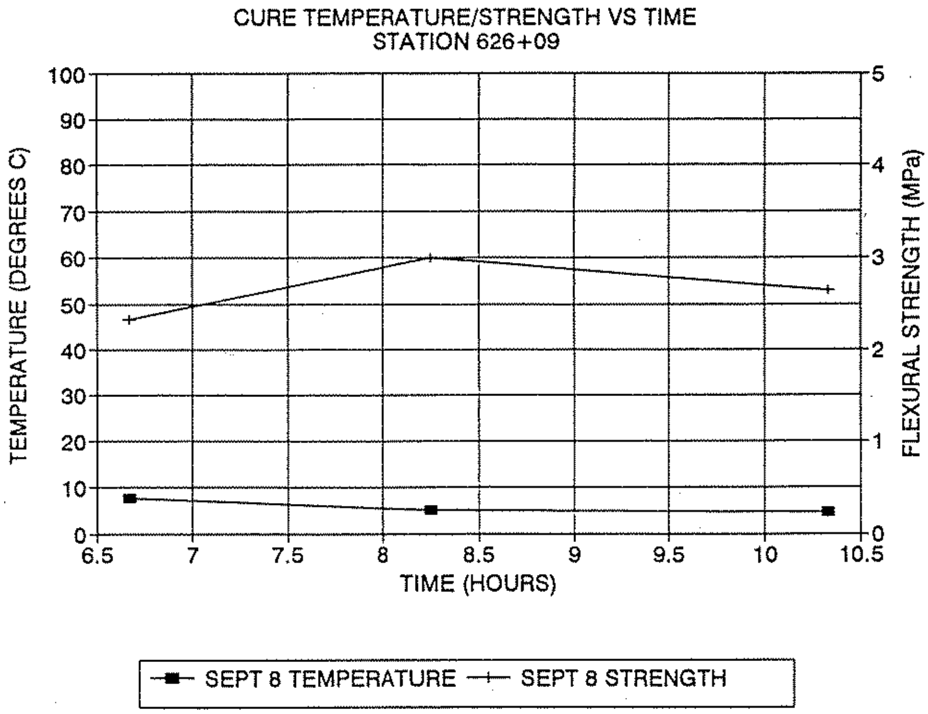




Figure D-1 Pulse Velocity, Station 618+00



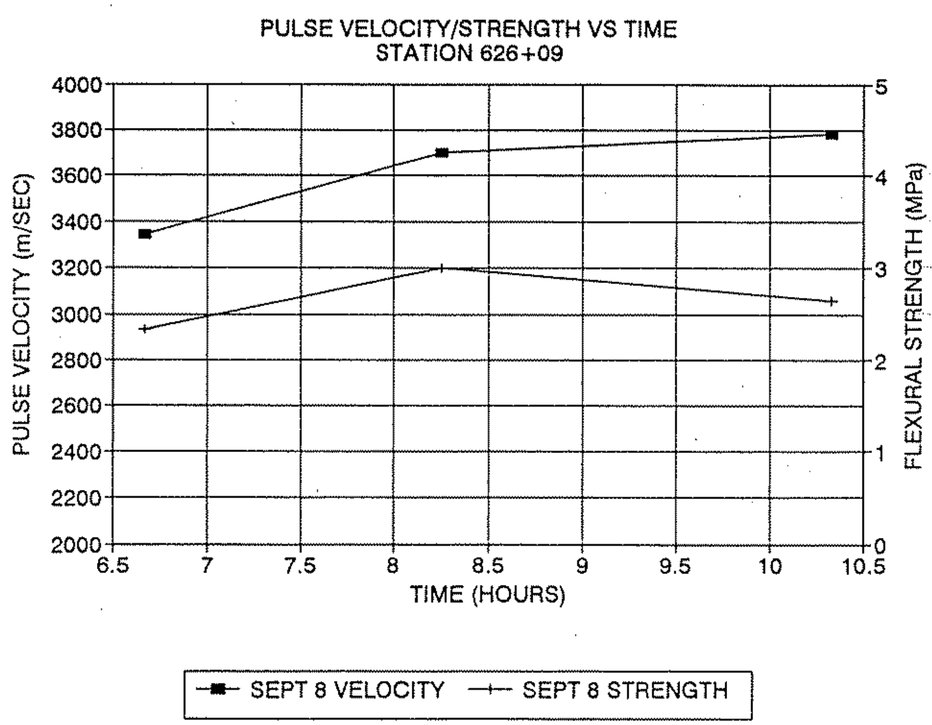
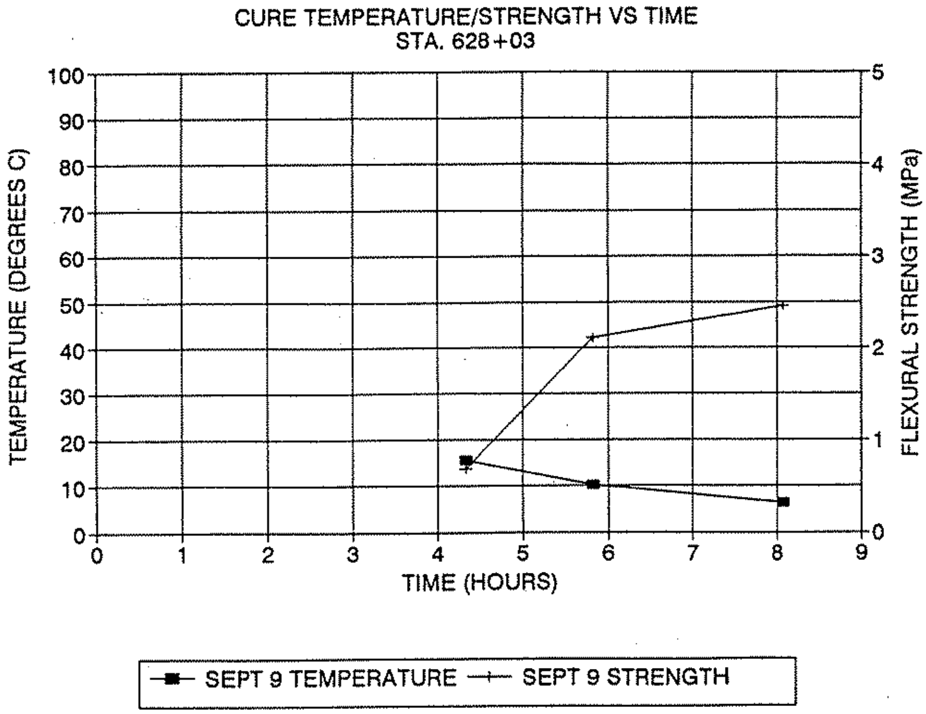




Figure D-2 Pulse Velocity, Station 626+09



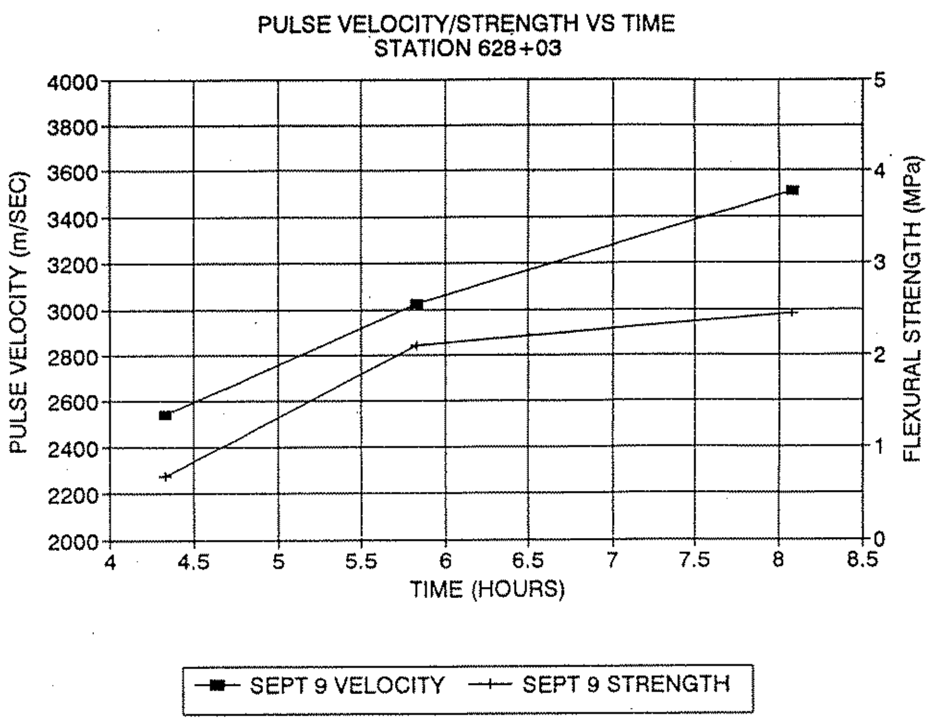
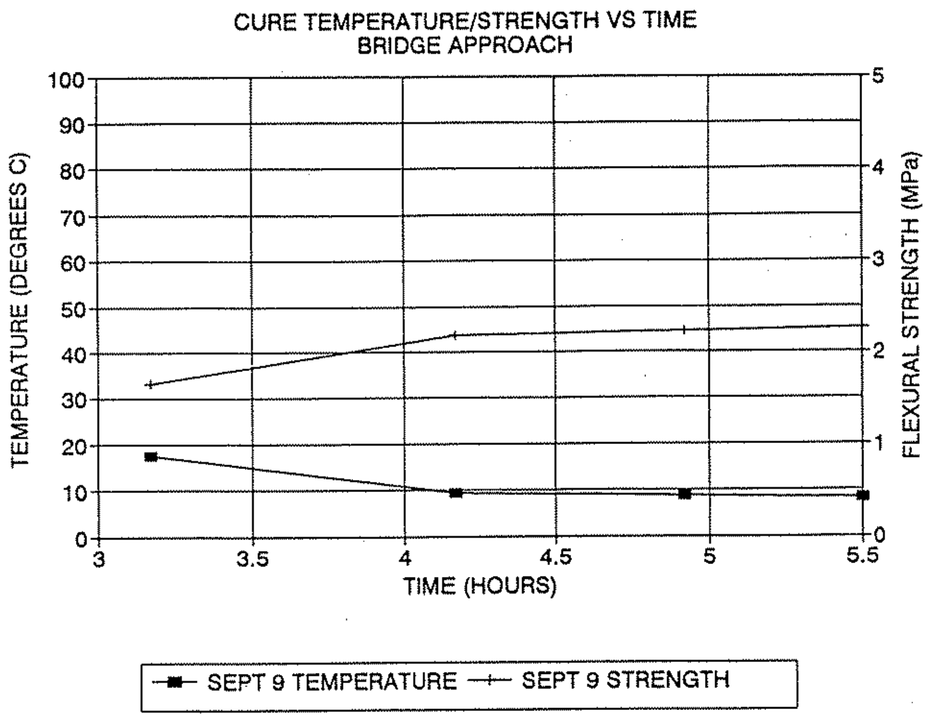




Figure D-3 Pulse Velocity, Station 628+03



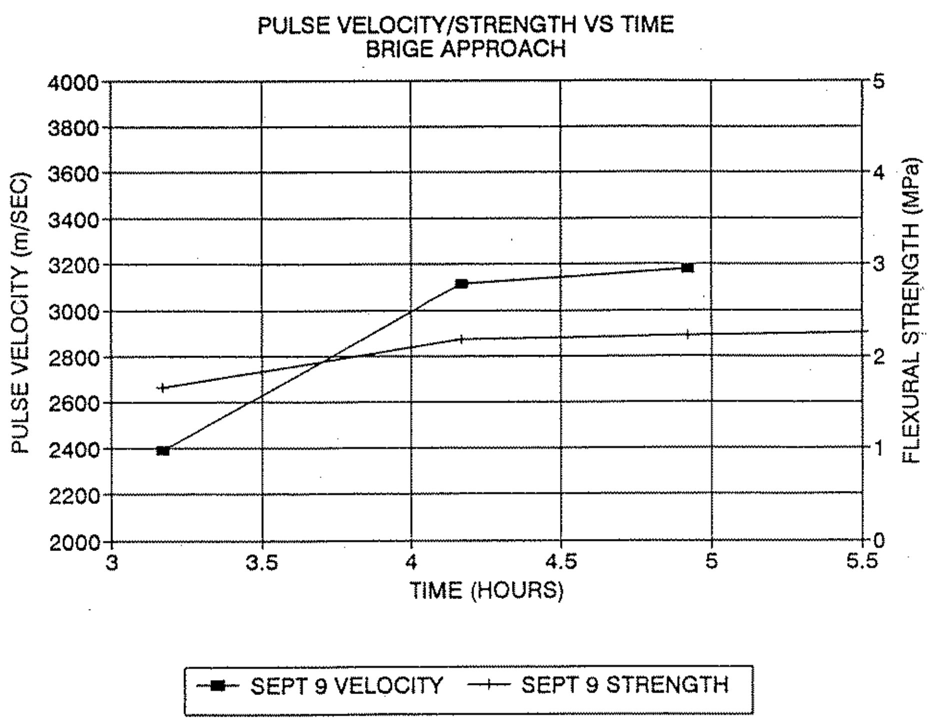
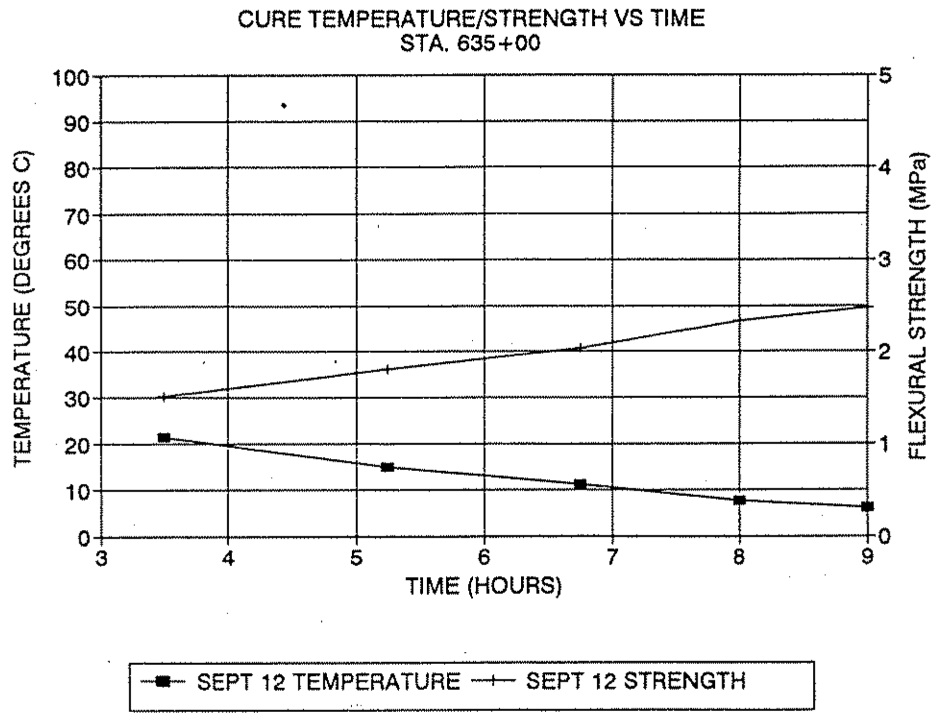




Figure D-4 Pulse Velocity, Bridge Approach



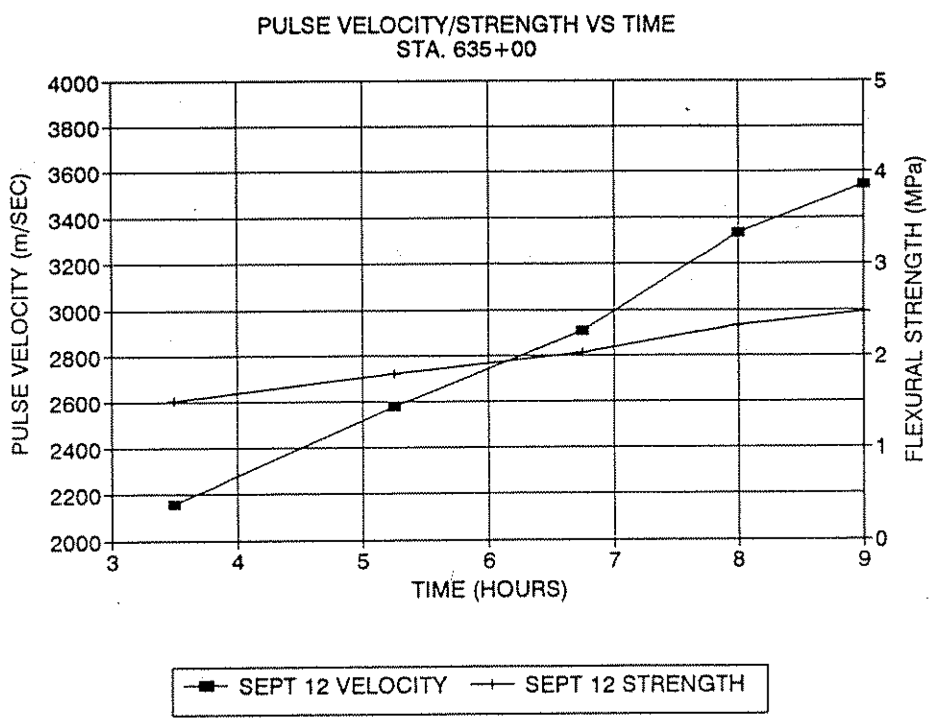
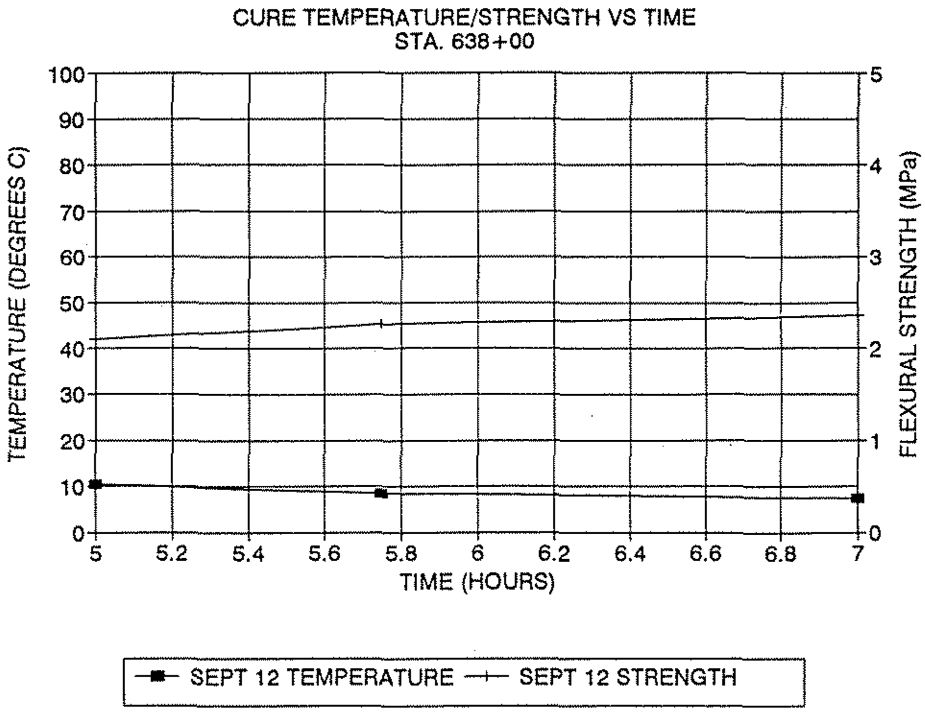




Figure D-5 Pulse Velocity, Station 635+00



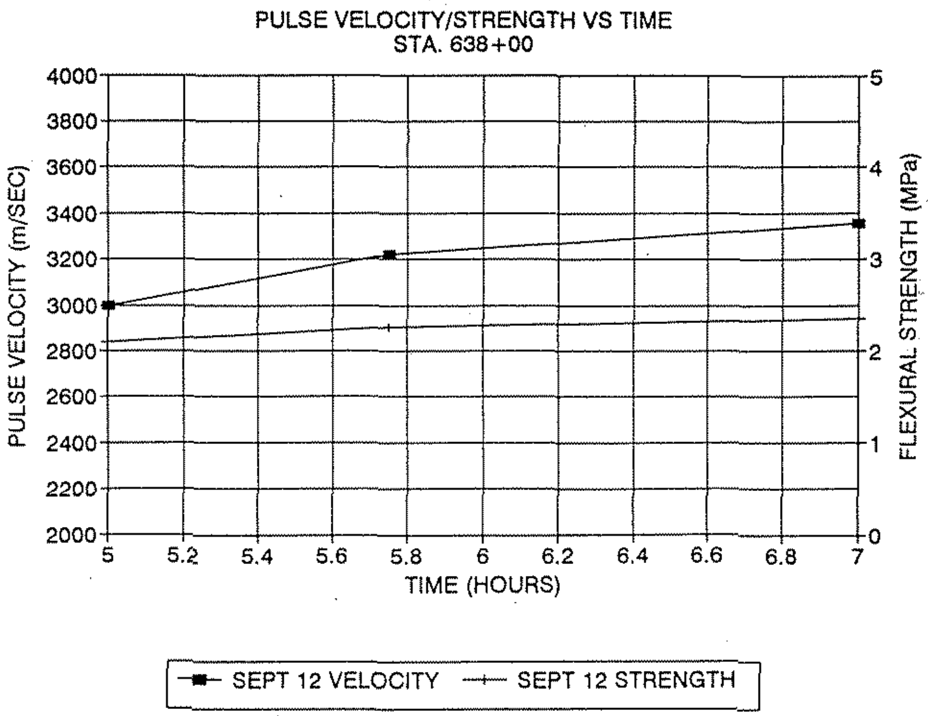
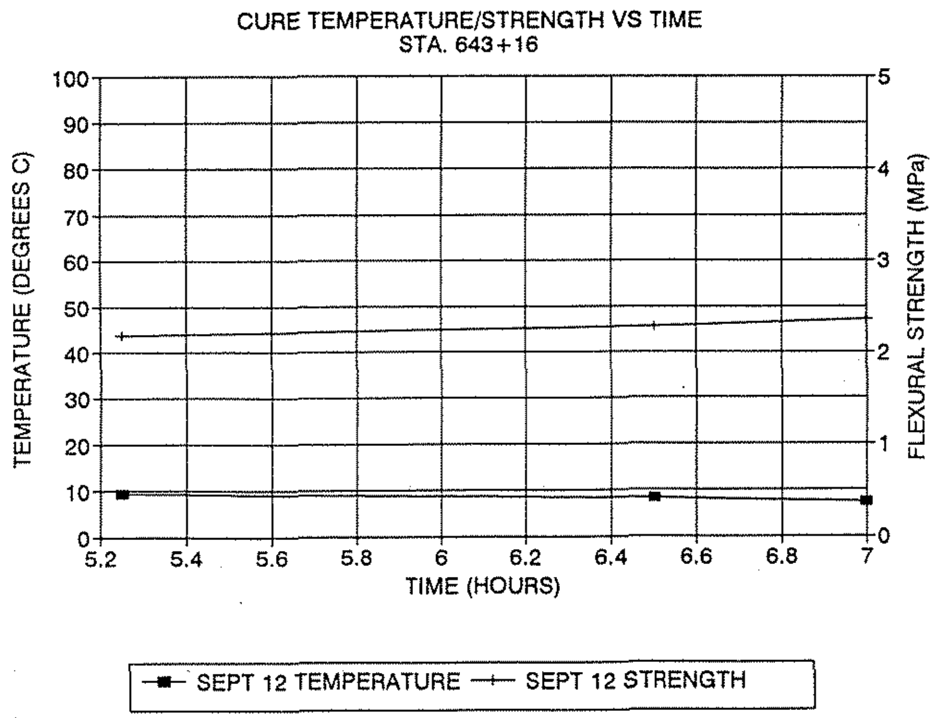




Figure D-6 Pulse Velocity, Station 638+00



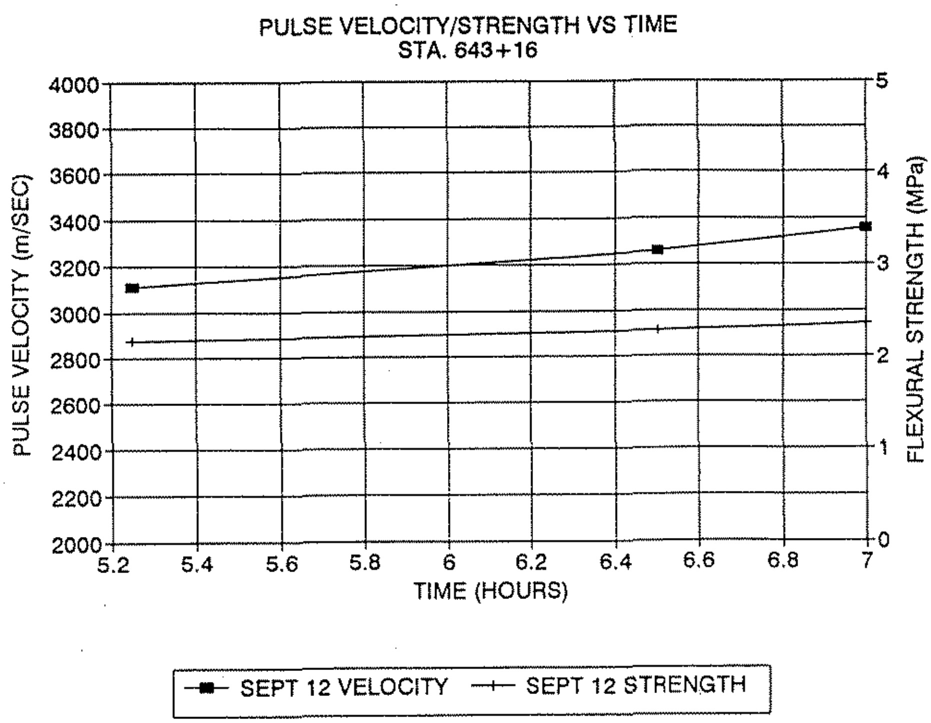
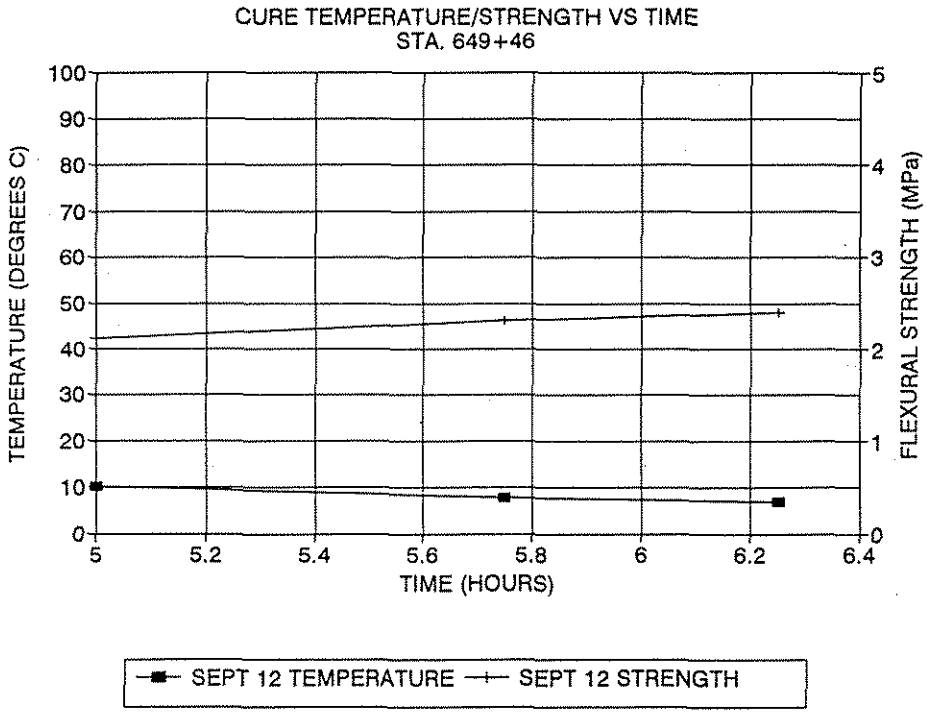




Figure D-7 Pulse Velocity, Station 643+16



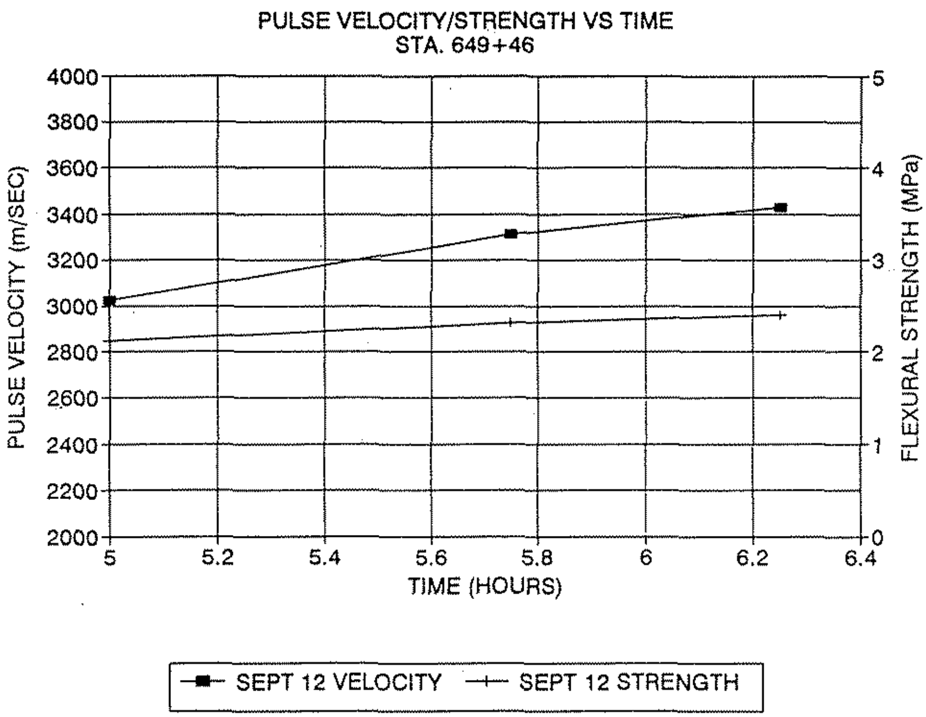
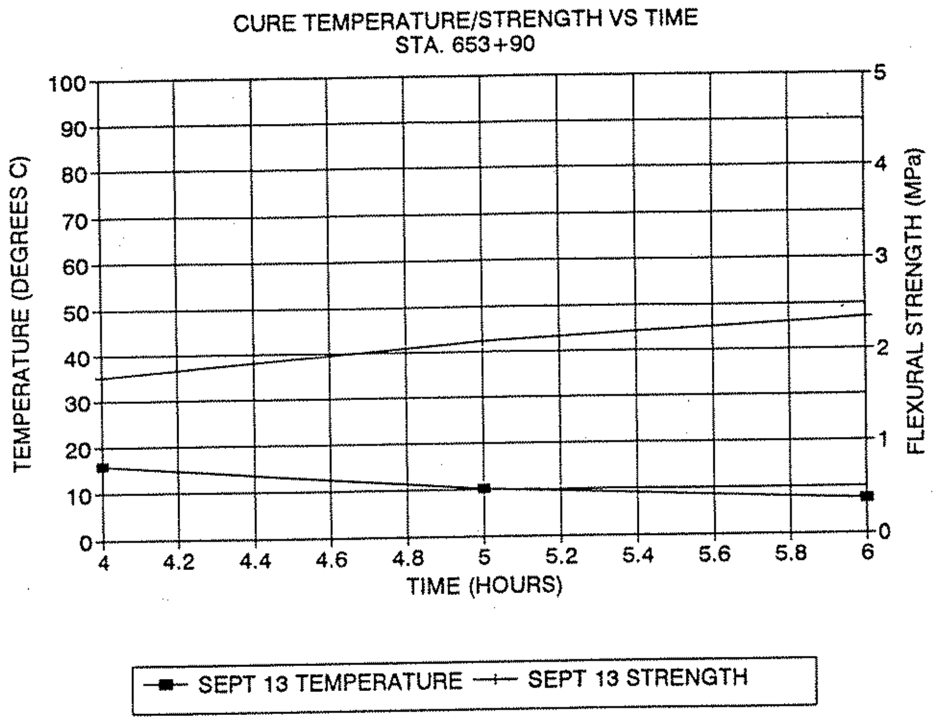




Figure D-8 Pulse Velocity, Station 649+46



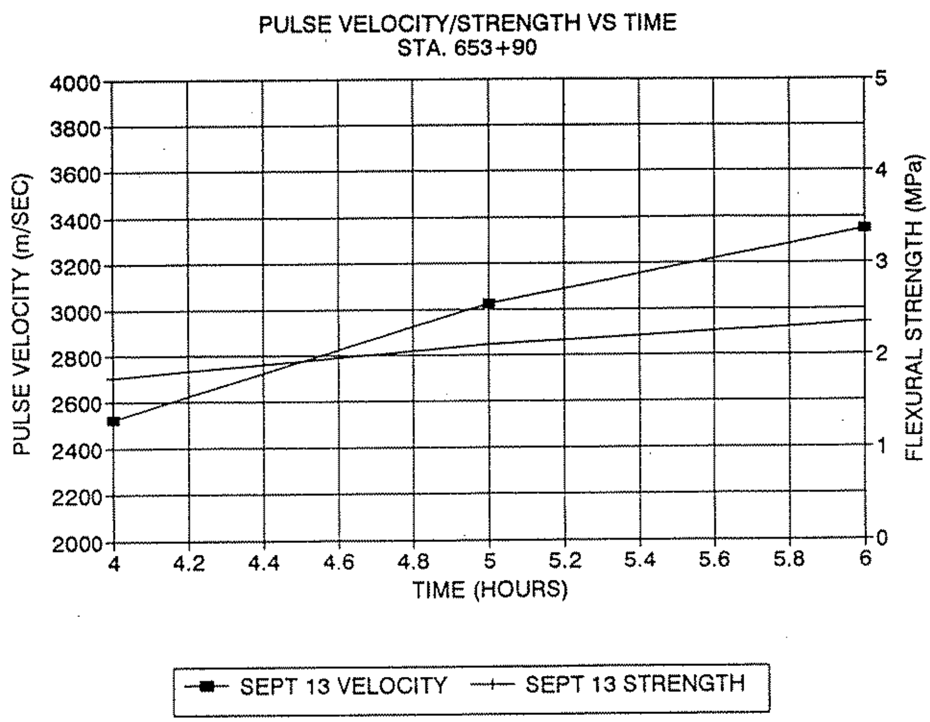
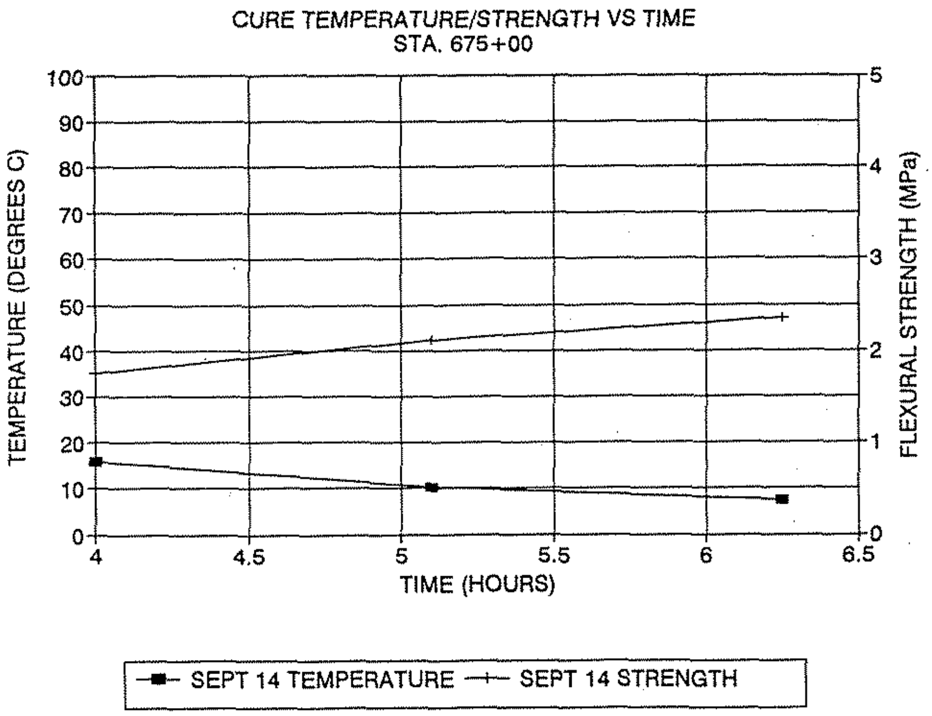




Figure D-9 Pulse Velocity, Station 653+90



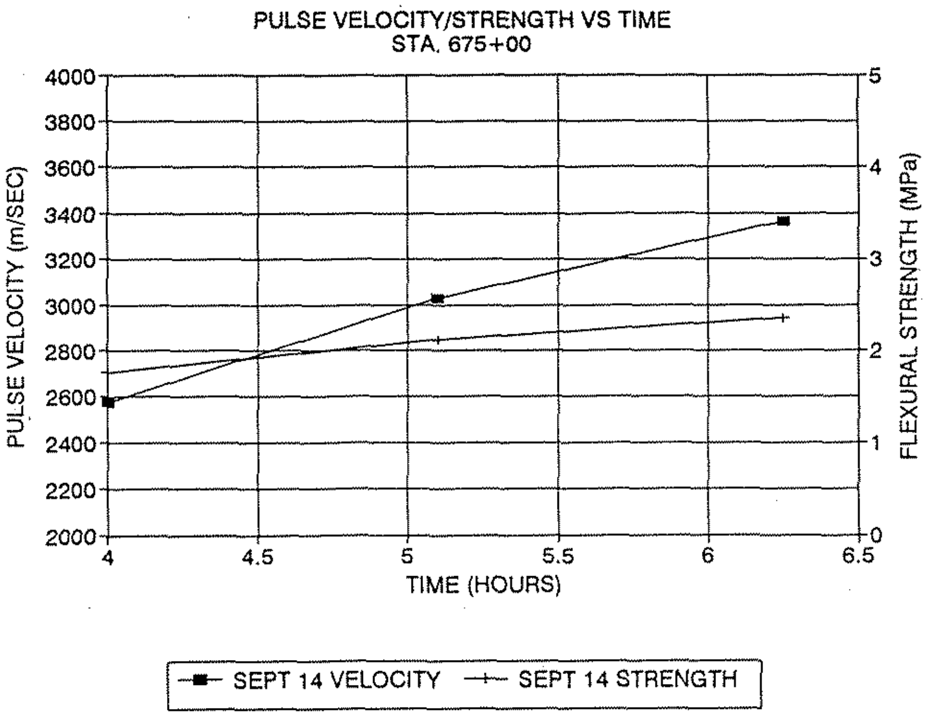
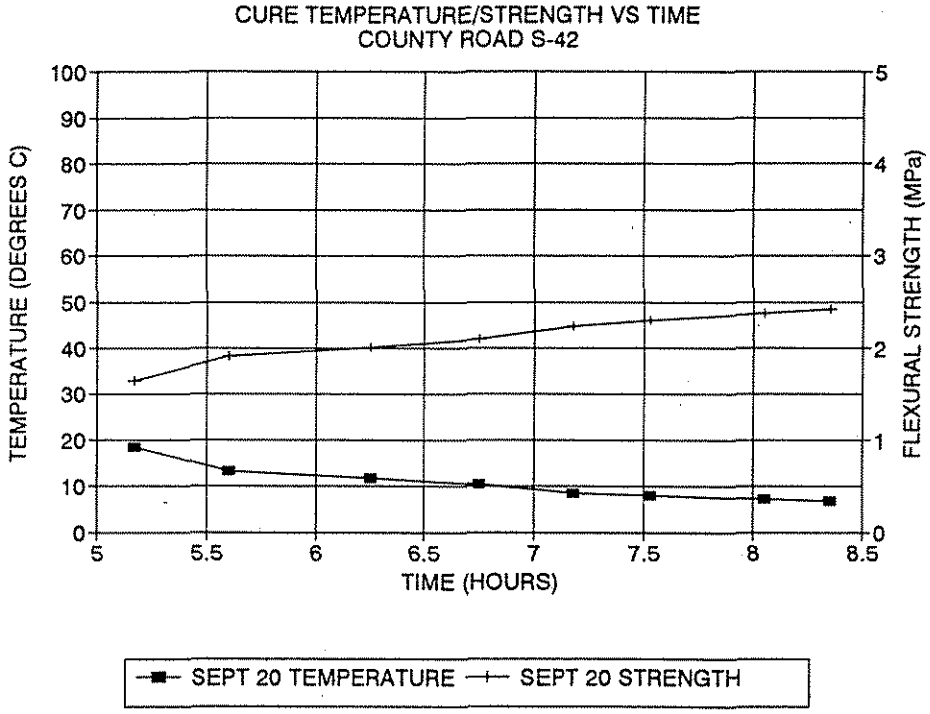




Figure D-10 Pulse Velocity, Station 675+00



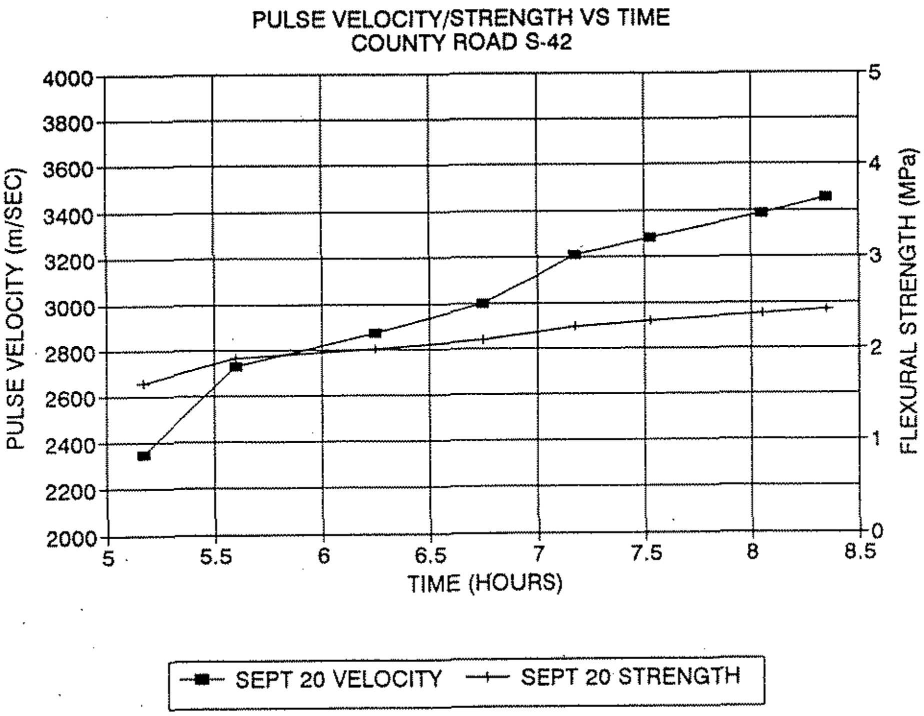
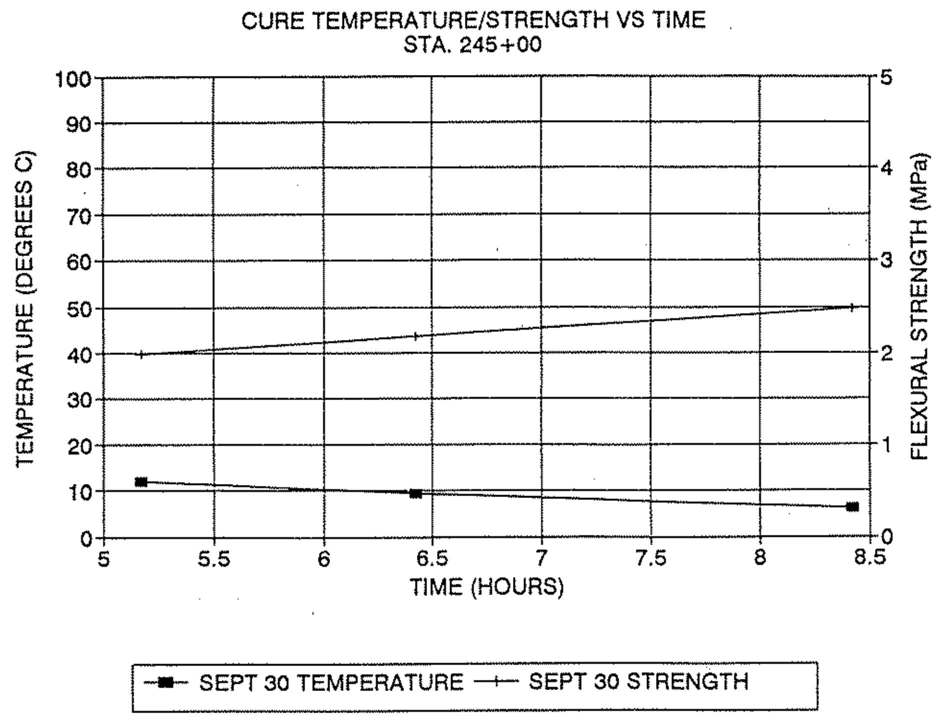




Figure D-11 Pulse Velocity, County Road S-42



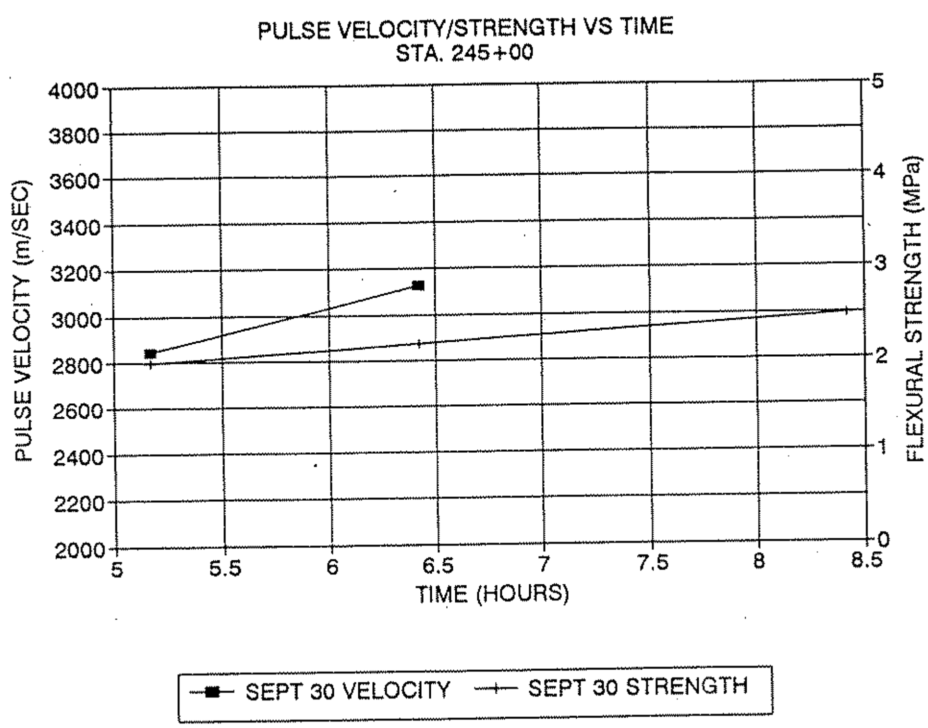
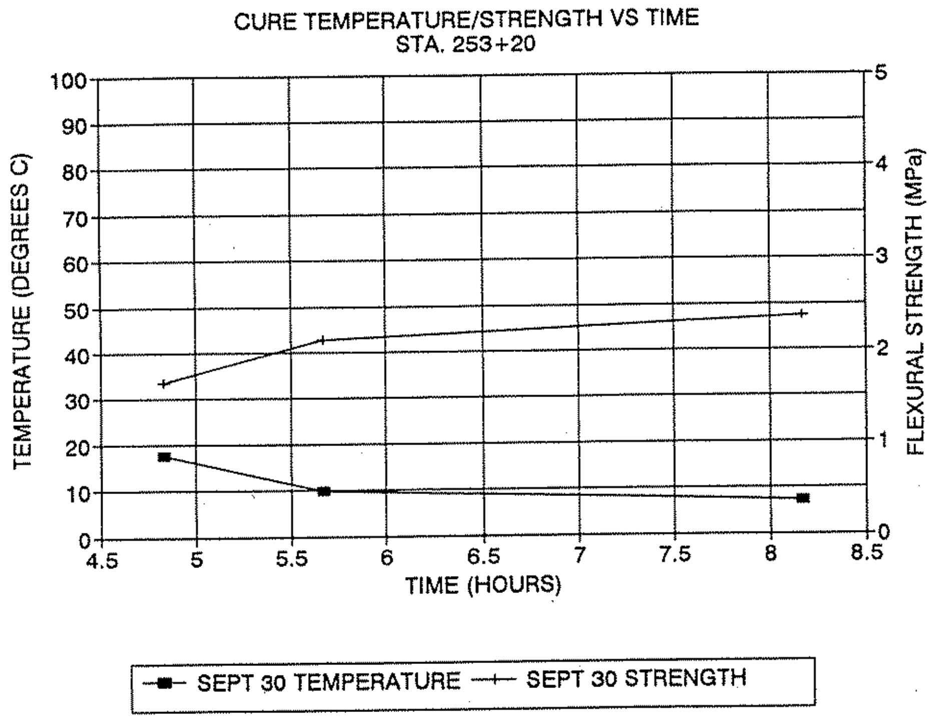




Figure D-12 Pulse Velocity, Station 245+00



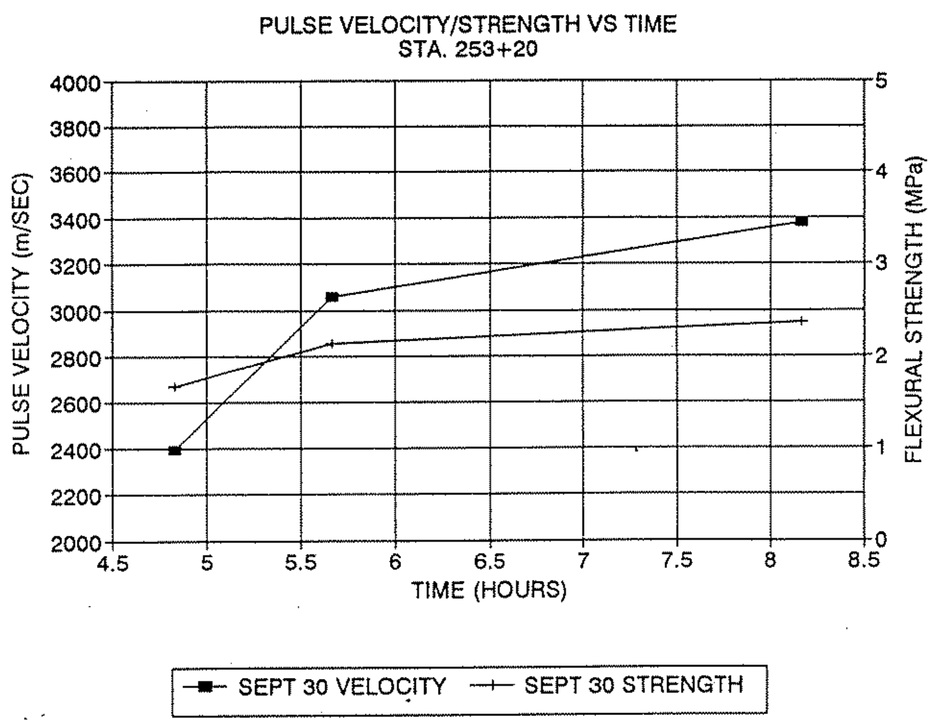
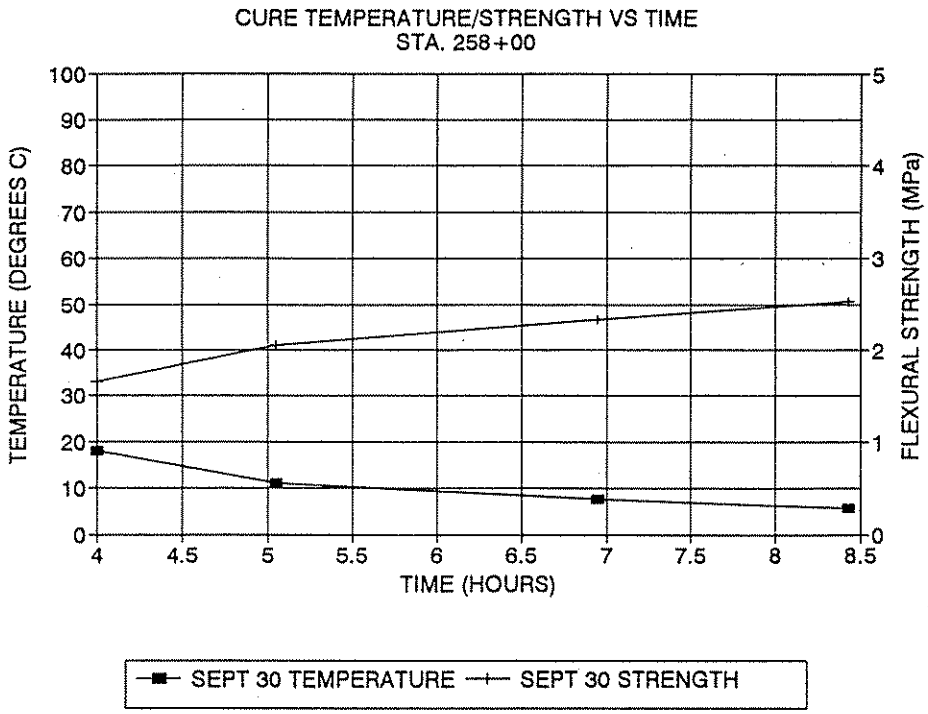




Figure D-13 Pulse Velocity, Station 253+20



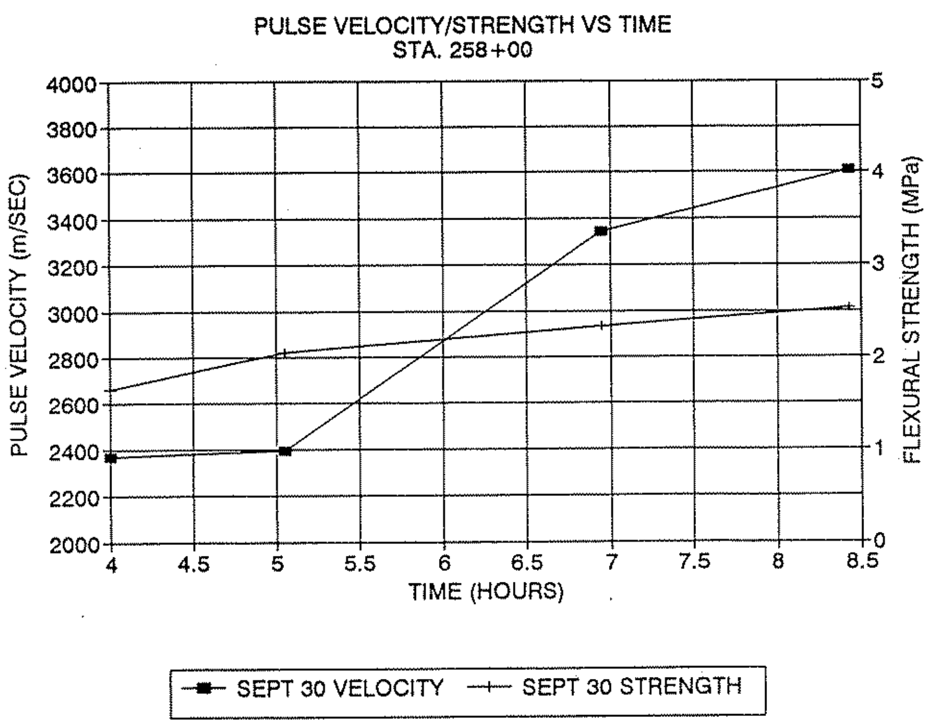
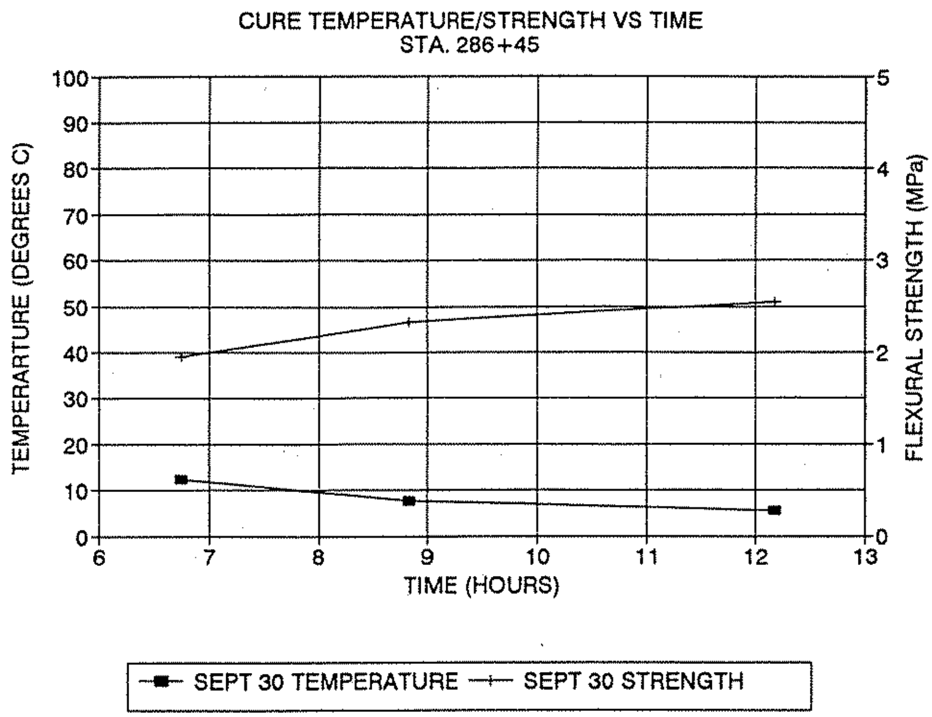




Figure D-14 Pulse Velocity, Station 258+00



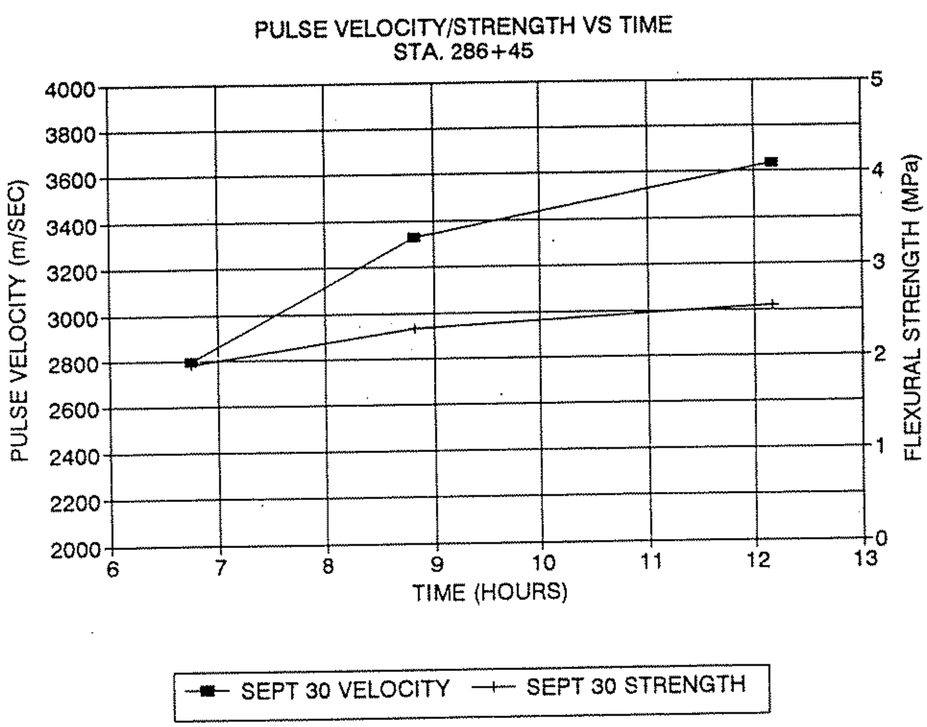
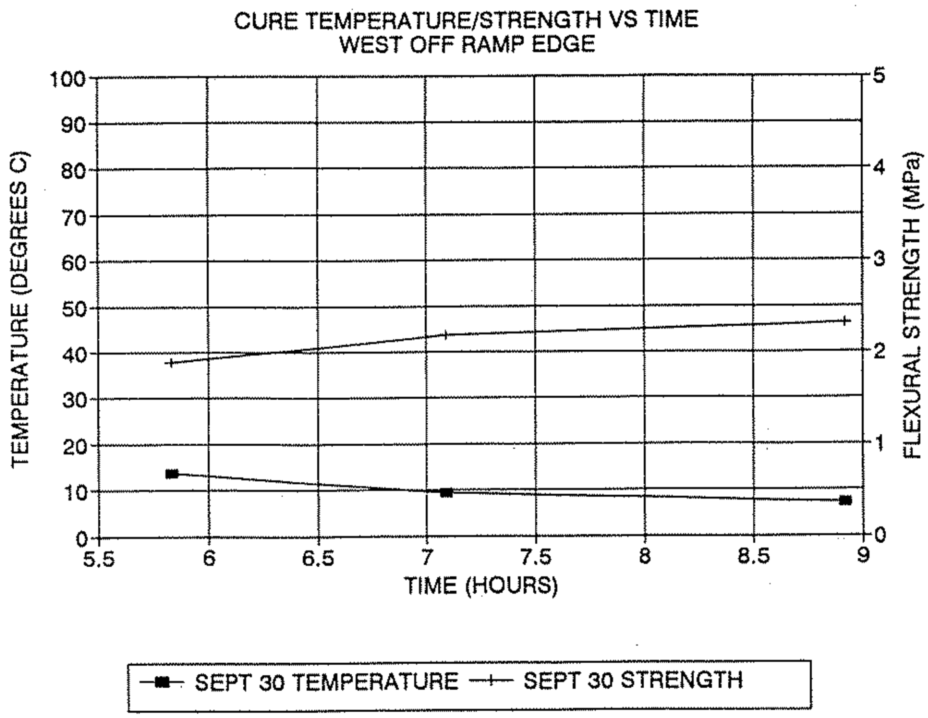




Figure D-15 Pulse Velocity, Station 286+45



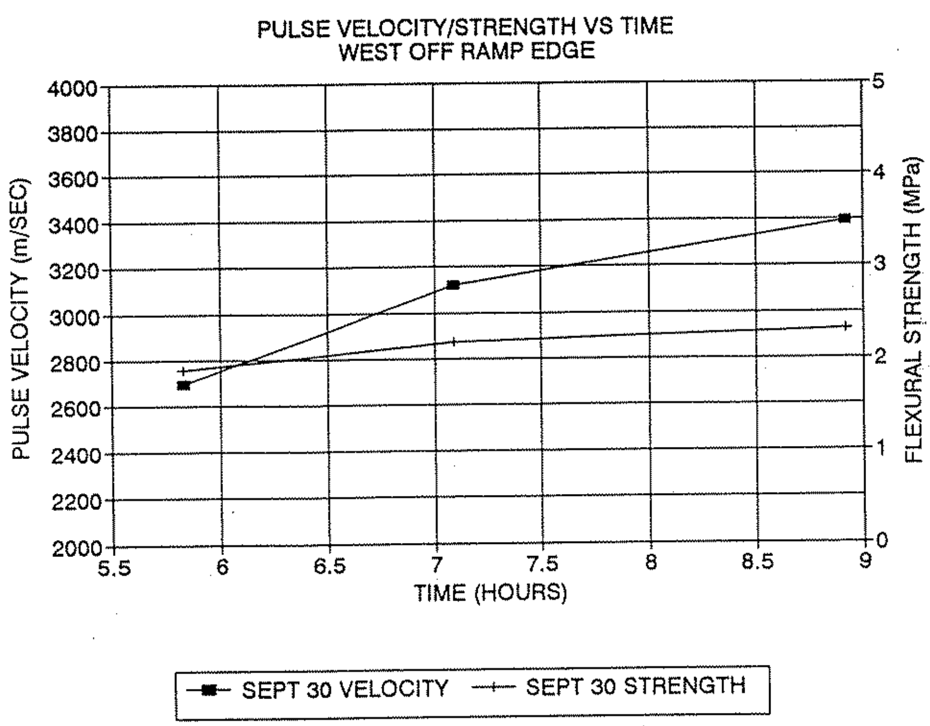
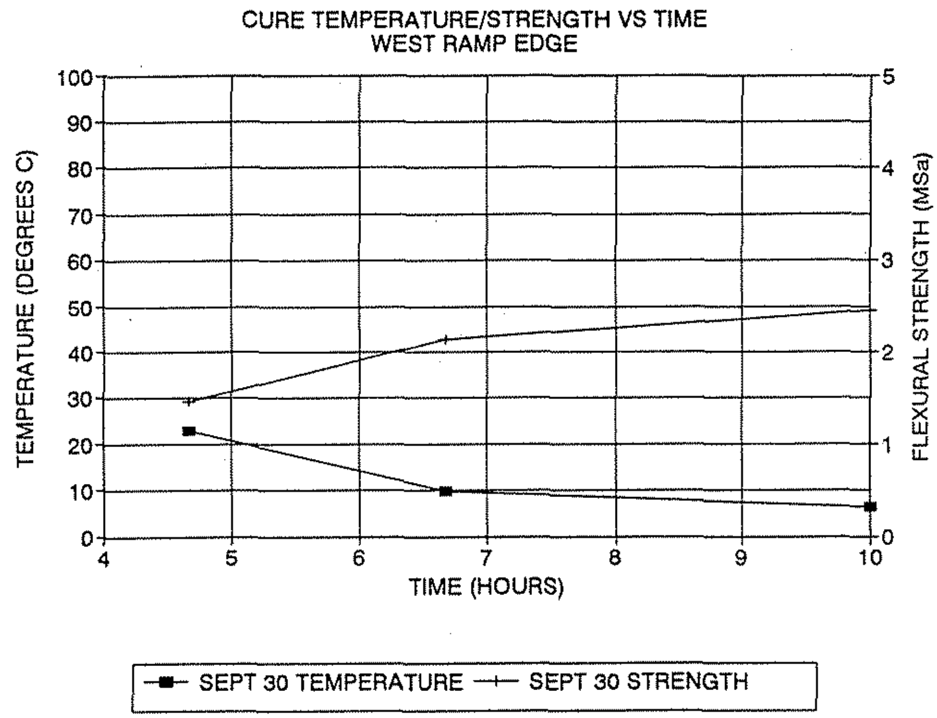




Figure D-16 Pulse Velocity, West Off Ramp Edge



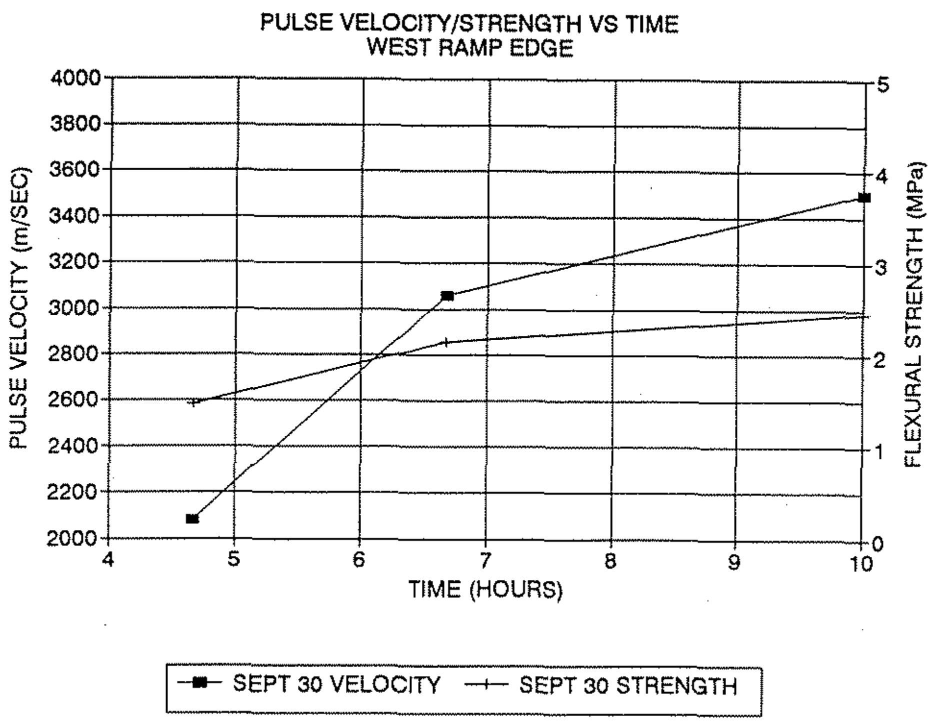




Figure D-17 Pulse Velocity, West Ramp Edge

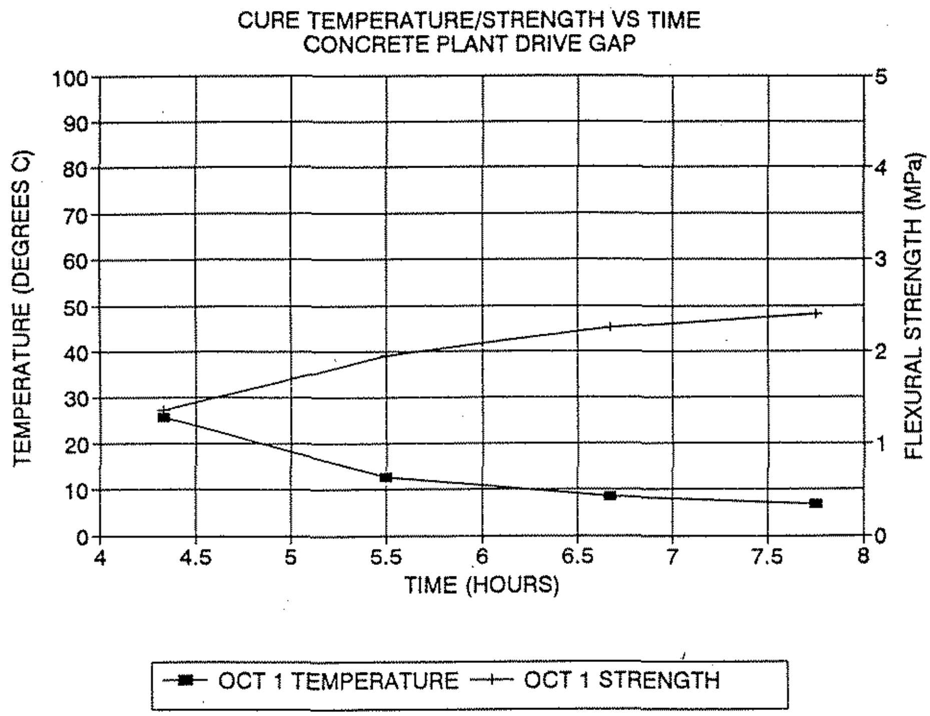



Figure D-18 Pulse Velocity, Concrete Plant Drive Gap

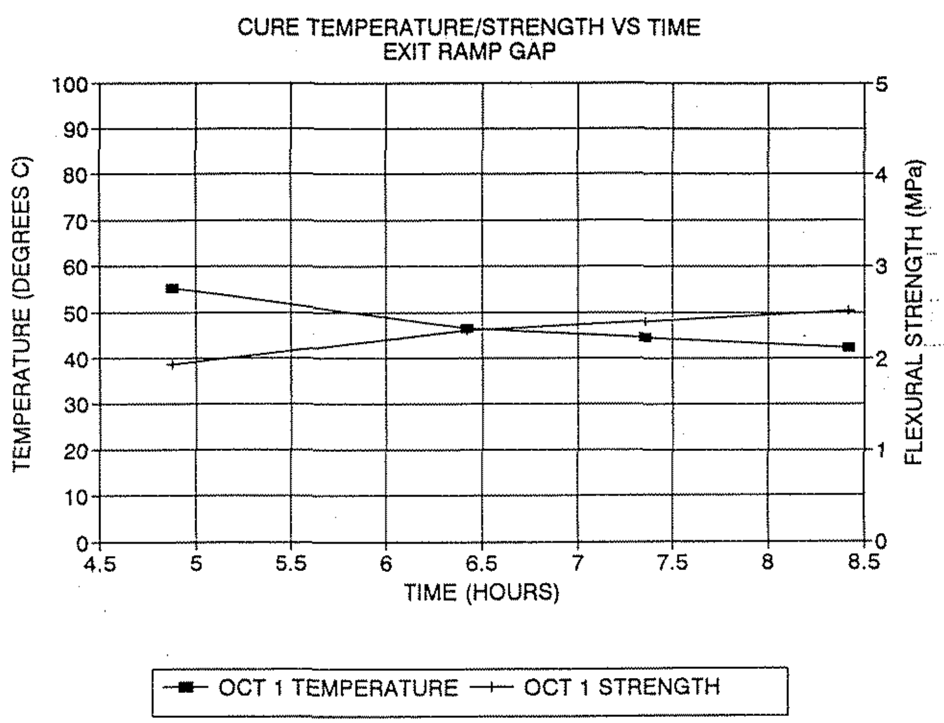



Figure D-19 Pulse Velocity, Exit Ramp Gap

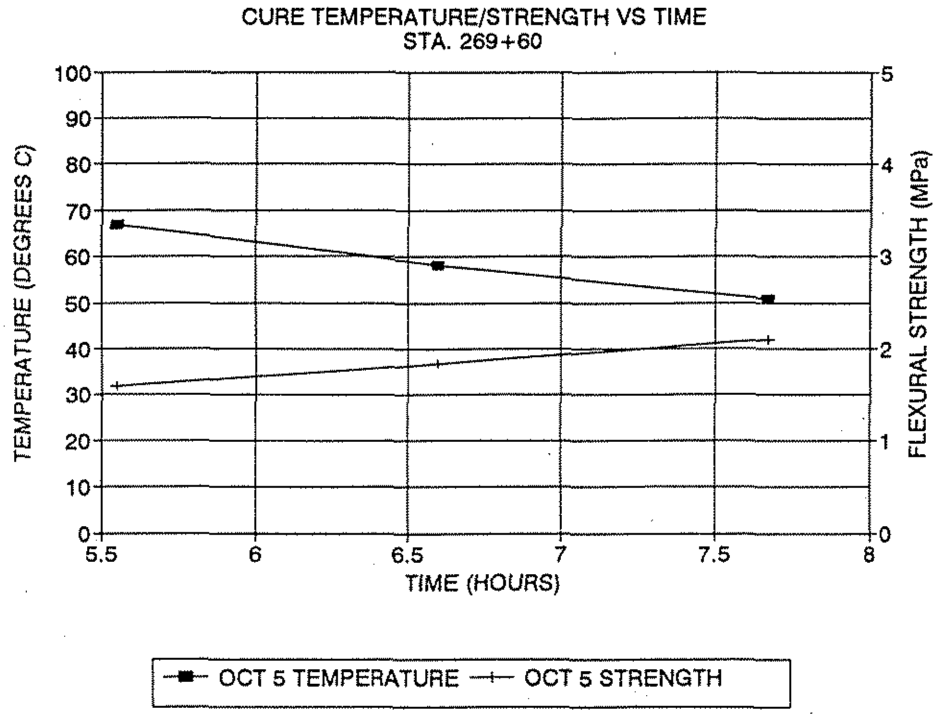



Figure D-20 Pulse Velocity, Station 269+60

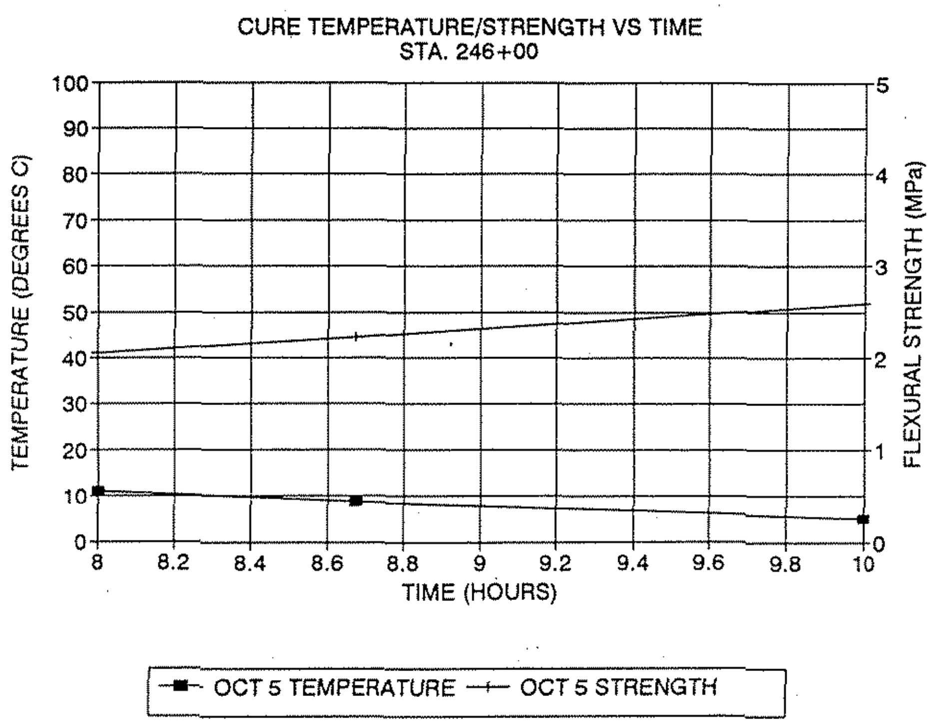



Figure D-21 Pulse Velocity, Station 246+00

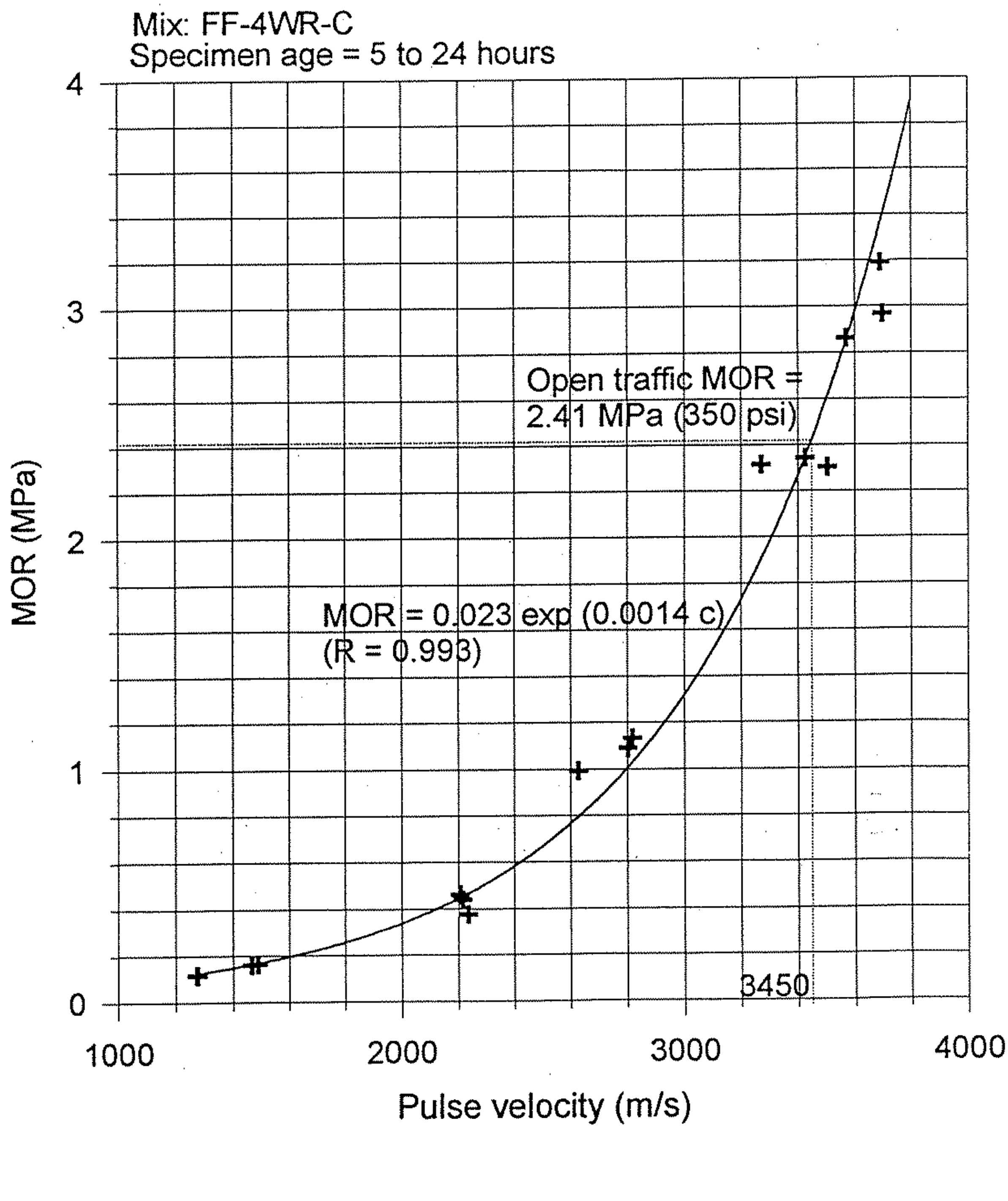
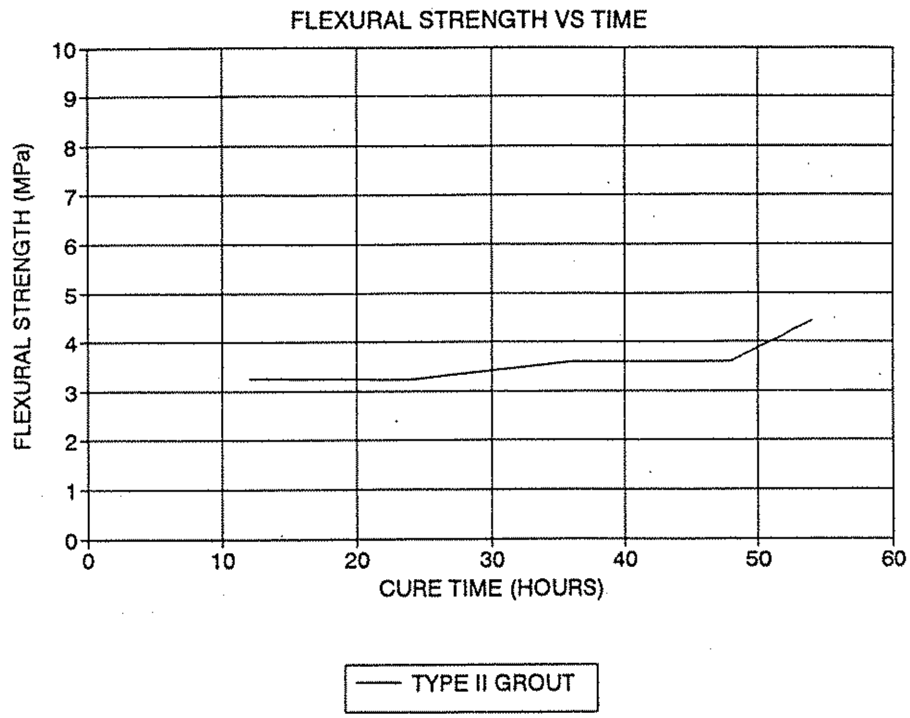



Figure D-22 Relationship Between Pulse Velocity and MOR (Strength)

## APPENDIX E



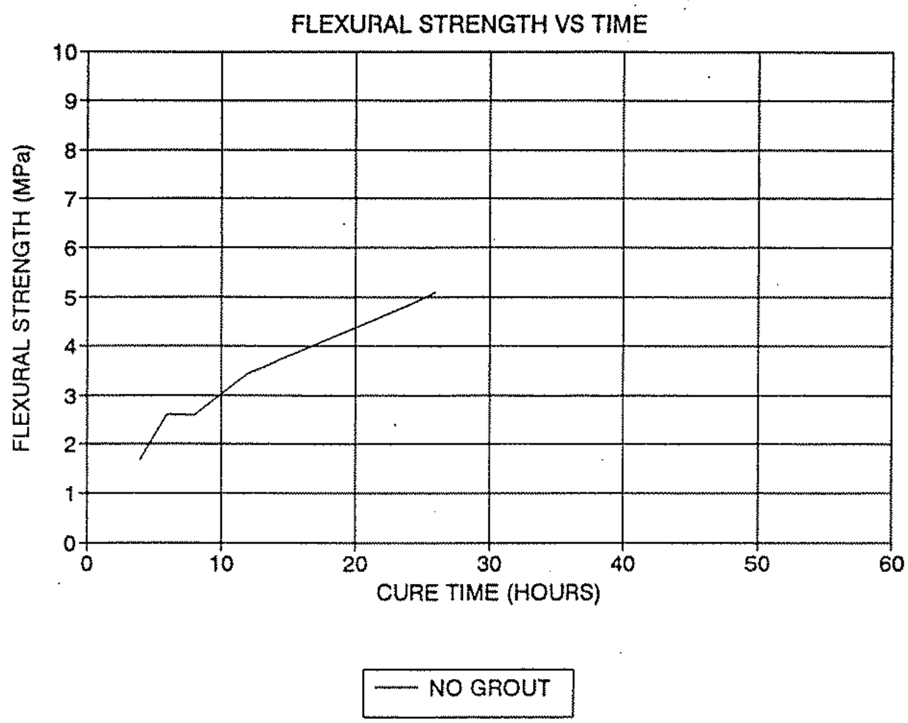




Figure E-1 Flexural Strength Test Beam Results

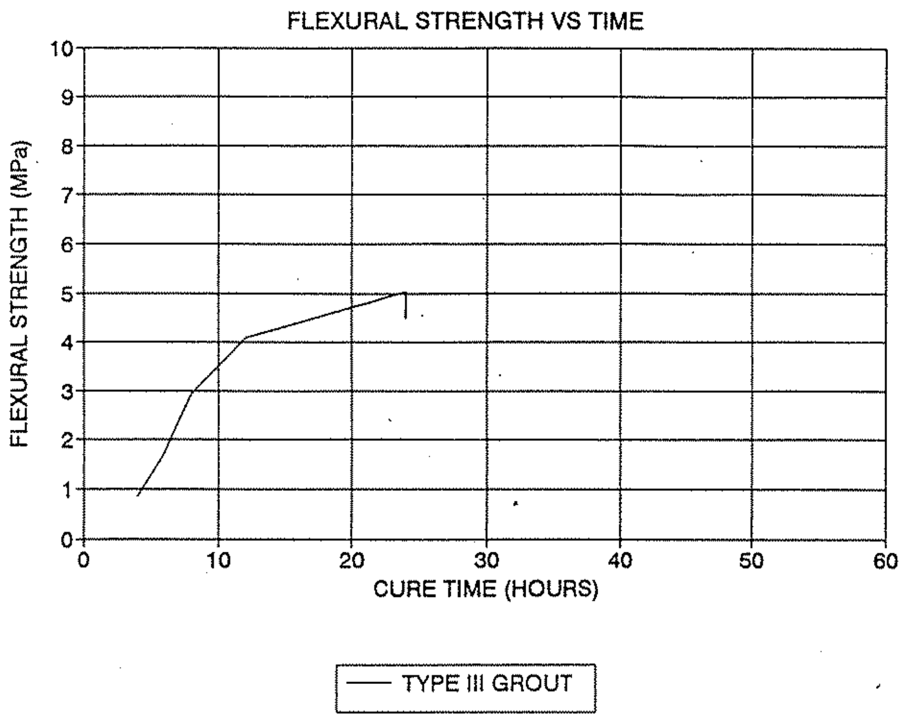



Figure E-2 Flexural Strength Test Beam Results (continued)