Research Report

KTC-90-9

INSTRUMENTATION OF THE TWELVEMILE BRIDGE

by

Theodore Hopwood II
Transportation Research Engineer

David Q. Hunsucker
Transportation Research Engineer

and

Edgar E. Courtney
Research Technologist

Kentucky Transportation Center
College of Engineering
University of Kentucky
Lexington, Kentucky

in cooperation with
Transportation Cabinet
Commonwealth of Kentucky

April 1990
(Revised June 1992)

CONTENTS

EXECUTIVE SUMMARY

An experimental precast, post-tensioned concrete segmental girder bridge over Twelvemile Creek in Campbell County was instrumented with stress and strain sensors. Those sensors were read during beam fabrication and the initial portion of construction. The work was performed to address Transportation Cabinet questions about structural performance and also to assess the effect of long-term creep. The study and bridge were not completed as originally designed due a beam failure during post-tensioning.

A variety of stress, strain, and deflection measuring techniques were to be employed during the study. Several of those including concrete stress and strain meters, and vibrating-wire strain gages were embedded in beams and one pier cap. Those sensors were read during several phases of fabrication and construction.

Concrete maturity meters were also employed to determine temperature-time relationships with increases in curing strength for concrete used in the pier caps and in one beam. Compressive strength and modulus tests were also performed on test cylinders obtained from the pier and beam concrete.

This report describes the progress of the field instrumentation and laboratory work to the termination of the study. The study was terminated due to the failure of a beam during erection. The bridge was completed using conventional steel girders.

Based upon the limited study findings and the beam failure, recommendations are provided that the Transportation Cabinet: 1) conduct further research related to the use of maturity meters, and 2) consider employing nondestructive testing of similar beams in fabrication shops.

INTRODUCTION

In 1988, the Kentucky Transportation Cabinet approved the design and construction of an innovative segmental bridge to be constructed on the Ashland-Alexandria (AA) highway spanning Twelvemile Creek in Campbell County. The bridge was designed by American Engineering Company and was to be constructed by the Haydon Bridge Company. Completion was initially scheduled for November 1989.

The bridge was to be a three-span continuous girder structure having two 145-foot side spans and a 182-foot main span. The deck was to be 82 feet wide with four 12-foot lanes, two 10-foot shoulders, and a 14-foot median. The superstructure consisted of a 7 1/4-inch thick deck slab cast composite with seven continuous precast post-tensioned concrete (PPC) I beams, 90 inches in depth. The bridge was designed for an HS 25 loading.

Each continuous girder consisted of five 94-foot PPC beam segments. The center and pier-supported segments were to contain longitudinal post-tensioning strands. The end segments were to be simply prestressed, except at the segment splices. The other segments were to be post tensioned. The beam segments were to be spliced together by short post tensioning tendons at the end blocks cast in each segment.

The beam splices were formed to transverse diaphragms that laterally connected the beams. The diaphragms were post tensioned along their length.

That design was innovative in that it allowed use of PPC beams in bridge spans exceeding 100 to 110 feet, the typical limit for conventional prestressed-beam construction. The structure was to be erected using falsework and a new technique was developed for eliminating falsework on future bridges. That would enable more economical construction when spanning deep chasms. A mixture of lightweight and standard aggregates was to be used in the precast semi-lightweight (130 pcf) beams. Future designs employing even lighter weight concrete (115 pcf) offered the possibility for extending this design to spans exceeding 250 feet.

The Twelvemile bridge was considered to be innovative because of the following features:

1. semi-lightweight concrete PPC beams,
2. short beam splices for longitudinal connection of adjacent beam segments,
3. post-tensioned pier caps,
4. post-tensioned diaphragms,
5. transversely post-tensioned deck (using standard concrete),
6. segmental I-beam construction, and
7. 7,000 psi concrete in the beams (using a high-range water reducer).

TEST PROGRAM

Before construction of the bridge, Transportation Cabinet personnel decided that it would be desirable to instrument the bridge. Kentucky Transportation Center personnel were to accomplish that task. Long-duration stress, strain, and deflection measuring gages were to be cast in or placed on the pier caps, beams, diaphragms, and deck. As a cost-reduction measure, only a representative portion of the structure was to be instrumented based upon the application of quarter symmetry. A total of 128 gages of different types were to be used. Measurements were to be obtained at various stages of beam fabrication and field erection. A proof test was to be conducted to determine the load distribution across the deck and structure upon completion of construction. Measurements were to be obtained at three-month intervals over a 30-month period to gain a better understanding of time-dependant structural behavior.

American Engineering Company, the bridge designer, proposed to analyze and interpret the more than 3,800 measurements that were expected. The primary purposes of the analysis were as follows:

1. to confirm design concepts, procedures, and assumptions;
2. to answer questions posed by Transportation Cabinet officials about the behavior of this type of structure;
3. to determine the load distribution on the deck and structure (in comparison with an advanced load distribution program developed by the University of Kentucky Department of Civil Engineering); and
4. to determine the time-dependant behavior of the structure over an extended period.

A formal construction report was to be prepared shortly after completion of construction. Two memorandum reports were planned, one describing findings from the load distribution testing and the second documenting the long-term analysis.

Tasks

The majority of the test program was to address concerns about the post-tension splices, semi-lightweight concrete used in the beams, and relative shrinkage between the semi-lightweight concrete and standard concrete used in the deck. The work was separated into 11 tasks.

Task 1 was intended to verify the moment transfer across the short beam splices. Stresses and strains were to be obtained and compared with design data. The magnitude of the moment across the splices was also to be determined. To accomplish that task vibrating-wire strain gages were to be placed in abutting beam segments. Three gages were to be placed at fixed depths in a beam in the end block and a fourth gage was to be positioned in the deck above those gages. Eight gages were to be employed at each of four test sites.

Task 2 was planned to show that a tension condition did not exist in the through-thickness direction of a beam (that no splitting problems existed in the end blocks). Two vibrating-wire strain gages were to be mounted transversely in beam end blocks at two test sites.

Task 3 was intended to verify that no shear (vertical displacement) occurred between the beams and diagrams. The beams were not keyed to the diaphragms and were held in place by the compressive force between the end blocks that connected the segments created by the short post-tensioning strands. Two Avongard crack displacement gages were to be used to measure displacements at four sites.

Task 4 was intended to measure stresses and strains in the beams and deck over a pier. That was to determine: 1) prestressing losses, 2) the effect of shrinkage between the deck and beam concrete, and 3) the effectiveness of the slab in assuming forces at the piers. Three Carlson strain gages and one Carlson stress gage were to be placed in two beams mounted over pier 2 on the south end of the bridge. Carlson reinforced concrete strain meters were to be placed in the deck over the beams at three locations. Also, three foil strain gages were to be placed on wires of a post-tensioning strand that ran in a duct through the beam.

Task 5 was planned to determine the long-term increase in deflection of a fascia girder due to creep. Ten optical surveying targets were to be attached to the west-facing fascia girder along the mid span and side span at the south end of the bridge. Surveying control points for measuring the deflections were to be established on the south bank of the creek.

Task 6 was intended to measure prestressing losses and determine the effect of shrinkage between the beam and deck at the midpoint of the main span. Three Carlson strain meters were to be placed in beams at the midpoints of two center segment beams. Those were placed at various depths in the beams. Also, Carlson strain meters were to be cast into the deck above the beam gages. Three foil strain gages were to be placed on wires of a post-tensioning strand that ran in a duct through the beam.

Task 7 was planned to determine live-load strains in the beams and deck and the load distribution across the deck. Two foil strain gages were to be mounted on the bottom flanges of several beams at their midpoints on the south side span. The gages were to be installed at two test sites.

Task 8 was intended to measure the strains induced in beam 4 during post-tensioning of the deck. Four foil strain gages were to be mounted on that beam at two sites, at the mid- and quarter- points on the south side span.

Task 9 was intended to measure the effective modulus of the cantilevered post-tensioned pier cap and the loss of post-tensioning due to creep. Three Carlson strain meters and one Carlson stress meter were to be installed at each of two sites on pier 2.

Task 10 was intended to measure the strain distribution along the deck slab. Six foil strain gages were to be bonded to the bottom face of the deck slab at two sites between beams on the south side span.

Task 11 was intended to measure the strength of concrete before post-tensioning. To accomplish that, concrete maturity meters were to be used to test four diaphragms, two piers, two beams, and the final deck pour.

<u>Instrumentation</u>

The Carlson strain and stress gages were to be used for direct embedment in concrete. Their function was to provide changes in strain between given periods and events. They provided readings that were to be corrected for changes in temperature. Those embedded gages also acted as temperature sensors. The gages were wired with 4-conductor, 16-gage wire. The wire was to be temporarily connected to a Carlson Model MA-4 Meter which provided readout of gage resistances that were proportional to strain. The gages were to be positioned in the castings by tying them to adjacent reinforcing steel with wires. The Carlson gages were selected due to their ability to make direct measurements in concrete and also due to their good stability for long-term tests.

Slope Indicator Company vibrating-wire strain gages were to measure strains in reinforcing steel embedments. They were mounted on machined reinforcing bars. Those bars were to be positioned in the forms by attaching them to conventional steel reinforcement. Those gages provided direct strain readings of the steel bars and indirect temperature measurements. Readings were to be obtained using a Slope Indicator Company Model 52669 vibrating-wire indicator. The readings could be compared with zero readings obtained during tests before installation. Further strain readings were to be obtained at the different stages of fabrication, construction, or service. The gages employed 2-conductor, 24-gage wire. The vibrating-wire gages were selected due to their good stability in long-term tests.

Several concrete maturity meters were to be employed during the study to predict increases in concrete strength during curing. The theory behind the use of maturity meters is that concrete curing strength is proportional to both curing time and internal curing temperature. If two concrete cylinders cured for equivalent time periods, but at different curing temperatures, the cylinder that cured at the higher temperature would have a higher compressive strength.

Maturity meter readings summing the effects of curing time and temperature are commonly stated in terms of curing time in hours relative to a fixed reference temperature, commonly 20° C. The resulting values commonly termed "M numbers" are typically expressed in equivalent 20° C-hours. For example, a concrete cylinder that cured at a temperature of 40° C for a period of 10 hours would provide an M number of 20 (in equivalent 20° C-hours). If the cylinder cured at a temperature of 10° C for the same time period, it would provide an M number of 5 (equivalent 20° C-hours). Two concrete cylinders containing identical concrete, but curing at different temperatures, would have equal strengths when each cylinder reached a specific M number.

James Instruments Model 3014 M-Meters were used. The meters employed temperature sensors (thermistors) embedded in curing concrete. The meters provided a running total of M numbers, curing time, and current concrete temperature on visual (LED) readouts on the face of the instrument. The meters could obtain and store readings from six separate sensors. The meters provided automatic concrete maturity readings that were updated hourly and stored by internal memory. Post-test printouts could be obtained of average hourly temperatures in the curing concrete.

The temperature-time relationships (M numbers) provided by the meters were to be correlated with strength increases in the concrete as it cured. That was to be done by periodically breaking concrete cylinders stored in the same environment as the concrete monitored by the maturity meters and correlating the resulting test strengths with the corresponding M numbers.

Conventional foil strain gages were to be employed to measure strains in the post-tensioning strands and on the exterior surfaces of the deck and beams. Measurements Group EA-06-062DN-350 Opt. E strain gages were to be used on the strands and EA-06-40CBY-120 gages were to be mounted on the beams and deck. The gages were to be connected to the strain indicator, a Megadac 2000, using 3-conductor, 26-gage wire. Extensive protection systems were to be used to protect the gages from moisture. It was anticipated that the gages would not be functional for more than 3 to 6 months.

The Avongard crack gages consisted of two pieces normally mounted on opposite sections between a crack. One piece of that gage is made from opaque plastic and has a grid scribed on its face. The other piece is transparent and has a cross hair scribed on its surface. The two pieces are mounted on the surfaces of the opposite sections with the cross-hair portion of the transparent piece centered over the grid in the opaque piece. Their relative movement can be monitored by measuring the movement of the cross hair in relation to the grid.

During the tests, a specific identification number was assigned to each gage. That number was used to identify wiring and to insure that readings were properly correlated with the correct gage. Care was taken to prevent incorrect gage labelling and placement during the tests.

MATURITY METER TESTS

The initial maturity meter tests were planned for the cap of the north pier (pier 1). The original intent was to prepare test cylinders from a batch of typical Kentucky Department of Highways Class A concrete. Sensors for the maturity meters would be attached to representative specimens. On December 14, 1988, concrete test cylinders were cast at the Reis Concrete Company in Alexandria. That company also supplied the pier concrete for the Twelvemile bridge.

The concrete specimens were water cured. All test cylinders including the instrumented control specimens were maintained at the same temperature. Compressive strength tests were performed on some specimens at intervals of 8 hours, and 1, 3, 7, 14, and 28 days. Those strengths were plotted against the maturity meter readings obtained at the same times (Figure 1). The resulting curve could be used to predict when the curing concrete had achieved a given compressive strength based upon a specific M number. The plot indicated a high increase in compressive strength during the first three days and a lower rate of increase in compressive strength thereafter. The three-day strength of the test cylinder concrete exceeded the minimum strength specified for form removal (3,000 psi).

The concrete curing strength results from those tests indicated the concrete would reach the form release strength in less than three days. If maturity meters were employed, the pier cap forms could possibly be stripped in two days if the large concrete pour, such as the pier cap reached an M number equivalent to the three-day values of the test cylinders.

When this was proposed, the bridge contractor was enthusiastic about the use of maturity meters. However, the maturity meters were not used to determine when the forms should be removed from the cap of pier 1. The contractor desired an extended working time for the concrete. He elected to use a set retarder which delayed the curing process. Also, the cap was cast Wednesday March 2, 1989 and the forms were not stripped for 5 days. Therefore, the previous KTC work and potential time savings were not utilized.

Many test cylinders were cast during the pour of that cap. Temperature sensors were placed in the top 3 inches of the pier cap at two locations. The one maturity meter monitoring the pier cap sensors was stored in a trailer at the job site. It was connected to the sensors by long lead wires. Readings were obtained at 1, 4, 5, 6, and 7 days. Test cylinders were instrumented with another maturity meter and taken to the KTC laboratory. They were water-cured and compression tested at 1, 3, 5, and 7 days.

In correlating the laboratory (test cylinder) M numbers with the field data from the pier cap, it was determined that the M numbers reflected the use of the retarder. The initial M number accumulation for the first 24 hours was low (23.9 equivalent 20° C-hrs). After four days, the effect of the retarder had diminished and the concrete cured rapidly as evidenced by the high M number (121.83 equivalent 20° C-hrs). The pier cap was in a cold environment at the time of the tests and the average job site temperature was about 0° C. The test cylinders were cured in water maintained at 20° C. That similarity was probably due to differences in ambient temperatures between the pier cap and test cylinders. The resulting maturity meter readings versus cylinder strength curves for laboratory tests of the concrete used in pier 1 are shown in Figure 2. The maturity meter readings versus curing time curves for the field tests of the concrete used in pier 1 are shown in Figure 3.

The contractor also elected to use the retarder when pouring the cap for pier 2. It was felt that the maturity meter data could not be employed to improve the progress of the work. No inside storage was available near the pier cap and the maturity meter was stored on top of the cap to discourage vandalism. It was wrapped in plastic to prevent water damage. Two temperature sensors were installed on the cap to monitor the curing temperature. The sensors were installed within the top 3 inches of the cap concrete. Test cylinders were cast, instrumented, and returned to the laboratory for curing. The resulting maturity meter readings versus cylinder strength curves for laboratory tests of the concrete used in pier 2 are shown in Figure 4.

A severe storm prevented access to the pier until seven days after the cap was poured. The maturity meter provided only one set of readings (114.1 and 120.3 equivalent 20° C- hrs respectively for the two sensors employed) and ceased functioning. The readings indicated the meter had ceased recording data six days after beginning the test. Moisture apparently entered the meter and caused a malfunction.

The last maturity meter readings were performed on beam 306 during steam curing at the Prestress Services of Kentucky Inc. fabrication shop at Avon on May 23, 1989. KTC personnel elected to test one beam with several sensors rather than test two beams with single sensors. The beam and two test cylinders were tested during steam curing. The beam sensors were placed in the top 3 inches of the concrete exposed at the top of the form. The sensors in the test cylinders were placed at a similar depth. The test cylinders were placed under the casting bed adjacent to the heating pipes.

The meters were read the following morning before the beam was removed from the casting bed. At that time, the beam had been steam cured for 15 hours. The maturity meter monitoring the cylinders was determined to be inoperable. The maturity meter monitoring the beam sensors provided M number readings of 86 and 107 equivalent 20-°C-hrs. The difference in those readings was probably due to the placement of the sensors. The lower reading was obtained from a sensor placed in a corner of the beam and the higher reading was from a sensor placed at its center. Test strengths of cylinders broken at the same time the maturity meter readings were obtained were 5,425 psi and 5,376 psi.

Both maturity meters were returned to the laboratory and hourly temperature data were obtained from their memories. The temperature data from the maturity meter monitoring the cylinders indicated temperatures 30° C lower than those encountered in the beam. Data indicated that if that meter had properly recorded M numbers, they would have been much lower than those from the beam.

The inoperable meter was returned to the manufacturer for repairs. Beam 306 was the last beam cast and no further beam monitoring was possible. KTC personnel intended to use the maturity meter to monitor the recasting of beam 201 which broke during installation. That beam was not recast by the end of this study. The maturity meters were not used further.

PIER CAP INSTRUMENTATION

The stem of pier 2 was cast upon completion of the concrete work on pier 1. The contractor placed two plastic conduits in the stem running upward to a location above the top of the cap. The conduits exited the stem at ground level where two electrical boxes were placed to house the wires from the gages. The reinforcing steel and forms were placed for the pier cap. Before pouring the cap, KTC personnel placed three Carlson strain meters and one Carlson stress meter at each of two locations in the cantilevered portion of the pier three feet from the stem. The strain meters were placed at various depths in the cap and the stress meters were located slightly below the post-tensioning ducts. The lead wires were routed to the top of the cap and into the conduit. The gages had been wired in the laboratory with pre-sized lead wires to facilitate their installation.

The pier cap was cast on March 28, 1989. KTC personnel monitored placement of the concrete to insure that the gages were undisturbed. The casting operations did not cause any problems. Baseline readings were obtained after the pour. The readings indicated that all gages were functioning properly. No further readings were obtained until the post-tensioning strands were adjusted.

The strands in the cap of pier 1 were completely post tensioned before similar work was begun on pier 2. The post tensioning caused the lower portion of the cantilevered arms of the cap of pier 1 to crack. KTC personnel assisted in monitoring the growth of those cracks by placing several Avongard crack gages across the cracks. The field consulting personnel were instructed on the use of those gages. They monitored the crack growth at those sites after the initial installation.

The decision was made to place extra reinforcing steel in the lower cantilevered portion of the pier 2 cap due to the cracking in pier 1. The strands were partially post tensioned until the beams were mounted on the pier cap. The partial post tensioning of the pier cap was performed on April 5, 1989. KTC personnel read the gages before and after the post-tensioning operation. The gages functioned properly and the readings were consistent with anticipated behavior.

No further readings were obtained.

BEAM INSTRUMENTATION

The beam instrumentation consisted of embedding Carlson stress and strain meters and Slope Indicator Company vibrating-wire gages in six beams (No.s 305, 306, 405, 406, 505, and 506). The beams were cast at the Prestress Services of Kentucky Inc. fabrication shop at Avon. The vibrating-wire gages had to be spot welded to flats milled into short sections of reinforcing steel. A vibrating-wire sensor had to be placed over the wire and the assembly had to be carefully sealed to inhibit moisture penetration. Short lead wires of various lengths were attached to the gages before installing the gages in the beam.

The gages were installed on the reinforcing steel framework after it was placed in the casting bed. No holes could be cut in the forms. Short leads were required between the gages and the exterior of the beams and ends of the leads were marked and routed to the face of the form. The free ends of those lead wires were coiled and wrapped in a plastic bag to prevent damage from the hot concrete during curing.

The fabrication shop was operating on a tight schedule and KTC personnel had to install the gages promptly. There was not sufficient time to check the gages for function. After the gage installations were completed, the form was assembled. Then, concrete was poured in the top of the form. KTC personnel could not effectively monitor the integrity of the gages during concrete placement. The gages had to be securely placed and the lead wires carefully strung on the reinforcing steel.

Prestress Services personnel typically placed the reinforcing steel and part of the formwork in the morning. KTC personnel installed the gages in the early afternoon. Prestress Services personnel then completed the formwork and cast the beam in the late afternoon. Steam curing was normally used. The curing was completed the following morning. Prestress Services test personnel then broke concrete test cylinders to determine whether the concrete had adequate strength. The steam curing was continued for several additional hours when strengths did not meet specified requirements. The forms were released and the beams were prestressed when further compressive tests indicated the required strength had been achieved.

The forms were stripped from the beam before removing it from the casting bed. KTC personnel explored the regions where the lead wires were originally located and chipped the concrete to expose the wires. The ends of the wires shifted in two cases during the concrete pour and could not be detected immediately. Several readings were not obtained due to the need to remove the beams promptly.

Baseline gage readings were obtained after the lead wires were located. The prestressing strands were released and another set of readings was obtained. The beams were then moved to a nearby storage yard and placed on blocks.

The rapid release of the prestressing strands created noticeable impact forces on the beams. Some vibrating-wire gages were located near those strands in the end blocks. Several gages functioned before the strands were released, but the impacts apparently damaged those gages and they did not function properly thereafter. The gages were placed away from the strands to reduce the effect of impact during later installations. No further failures were encountered.

The beams were observed to be very hot at the time of form removal and strand release. It was possible that most of the strain readings obtained at the casting bed might not be useful. Additional gage readings were obtained after the beams were cooled and placed on blocks. The gage placement on the beams began April 18, 1989 and ended May 24, 1989.

It became evident during the work on pier 2 that attaching lead wires from the beams to the panel boxes on the pier might prove extremely time consuming. KTC personnel felt that it would be better to attach all conduit and lead wires at the fabrication shop and limit the construction site work to running wires through the conduit.

KTC personnel sized, cut, and marked all necessary wires. Conduits were attached to the sides of the beams. They were connected to junction boxes that covered the points where the lead wires exited from the beams. The lead wires were spliced with the long leads necessary to reach electrical boxes mounted on the piers. Care was taken to ensure good electrical connections and proper insulation. The lead wires were placed through the conduits. Gage readings were obtained to insure that the gages functioned. The wires were looped and securely tied to reinforcing steel projections for transport to the job site. The wires were to be routed in the conduits after the beams were positioned. That would only require spanning diaphragms and pulling the wires through the conduits.

All optical gages were installed on the fascia beams (No.s 307, 407, and 507). Beam locations for foil strain gage installation were marked and surfaces prepared for gage installation. That required the application of epoxy to the surface of the concrete and follow-up sanding to remove all excess epoxy on the surface of the concrete. Wires were placed for the strain gage installations. No other work was performed on the completed beams.

Strain gaging of the post-tensioning strand wires was difficult. Practice gage installations were prepared. Corrosion protection presented problems. KTC personnel selected a rubberized coating, Measurement Group M-Coat J, with the additional covering of aluminum tape to prevent abrasion damage when the strands were pulled through ducts. Prestress Services personnel had placed styrofoam blockouts adjacent to the ducts at specific points in the test beams. The blockouts were to be removed after the beams were cast. Holes were to be cut into the ducts at those locations to provide exit points for the strain-gage wires. Work on the study was terminated before the strands were gaged and installed.

CONCRETE STRENGTH AND MODULUS TESTS

KTC and Division of Materials personnel performed compressive strength and modulus tests on the pier and beam concrete concurrent with the gaging of the pier cap beams. The test cylinders were cast at the respective job sites by technicians working for the consulting engineer (for the pier concrete) and the beam fabricator (for the beam concrete). The pier cap concrete cylinders were kept at the job site for several months and later transported to the KTC laboratory for outside storage.

Division of Materials personnel were to conduct the bulk of the compression strength and modulus tests at 16, 30, and 60 hours and 7, 10, 20, 40, and 80 days. KTC personnel were to perform those tests at 16 and 30 hours to confirm the Division of Materials results. The modulus tests were to be performed according to ASTM C 469-87a, "Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression." The strain readings were obtained using a compressometer during compressive loading of the test cylinders. The elastic chord modulus of elasticity is computed by the equation:

$$E = (S_2 - S_1)/(e_2 - 0.000050) \qquad (1)$$

where:

E = chord modulus of elasticity, psi;
S_2 = stress corresponding to 40 % of ultimate load, psi;
S_1 = stress corresponding to a longitudinal strain, e_1, of 50 millionths, psi; and
e_2 = longitudinal strain produced by stress S_2.

The modulus and compressive strength values for concrete from beams 305, 306, 405, 406, 505, and 506 tested by Division of Materials personnel are listed in Tables 1-6. Those tests are for concrete cured up to 80 days.

The general trend for modulus and cylinder strength data from both organizations was to increase with curing age. The Division of Materials values for the concrete from the six beams are plotted in Figures 5-10. The KTC strength and modulus data were plotted in Figure 11. In several cases, the Division of Materials data did not quite provide the anticipated strength or modulus increases. That was probably due to the use of single test values and to the anticipated variability in concrete properties. The general trend was as anticipated and supports the validity of the test results. The long-term test results by the Division of Materials indicates that the concrete would meet the final strength requirement of 7,000 psi.

DISCUSSION AND CONCLUSIONS

The study ended prematurely due to failure of a non-instrumented beam at the job site. The beam broke suddenly during post-tensioning operations and resulted in several fatalities. Work was halted and eventually Transportation Cabinet officials elected to replace the design with steel girders. A large part of the gage installation work had been completed at that time, yet very little useful information was obtained due to the status of construction. During this study, $49,605 had been expended by Kentucky Transportation Center personnel.

The KTC work had progressed satisfactorily. The gages that were installed in the pier cap and in the beams functioned as intended. All the pier cap gages and over ninety percent of the beam gages were determined to be functional after installation. Most of instrument readings scheduled prior to the termination of the study were obtained successfully.

Maturity meter data obtained during the study suggest that for heat-cured beams and large monolithic castings, test cylinders kept at the test site may not adequately reflect the time-temperature history of the actual structure. Test cylinder results including marginally rejectable strengths may not accurately reflect the strength of the structural concrete. Work may be delayed until additional test cylinders cure sufficiently to gain strength the structure already possesses. That retards the progress of construction and fabrication. Eventually, it results in increased costs for the Transportation Cabinet.

It may be desirable to develop a rapid heating system that would quickly cure specimens at a rate similar to steam-cured beams and advanced for ambient-temperature field pours. The quick-cured cylinders could be tested at specific time-temperature increments (M numbers). Once the critical strength is achieved, the corresponding M number from the structure could be used to determine when the structure or beam was ready for further processing such as form removal or post tensioning. A maturity meter monitoring the actual structure would indicate when the minimum acceptable M number was reached (based on the cylinder test results). This would enable contractors and fabricators to avoid delays in waiting for test cylinders to cure. Maturity meter temperature sensors would need to be placed in the coolest portion of the structure since the M-number should reflect the minimum strength.

Events related to the beam failure indicate the need for nondestructive inspections of post-tension beams. They are dependant upon several complex and interrelated factors including, concrete mechanical properties (strength and modulus), the presence of proper structural reinforcement, the location of reinforcement, and the geometry and sizing of the structure. No single conventional test can adequately ensure the structural integrity of a beam. The beam condition may be adequately characterized by its stiffness.

Dynamic testing, the measurement of the resulting deflection created by a small impulse force, may be a good method to determine beam integrity. The tests would be comparative among a family of beams. The test instrumentation would measure the relative stiffness of the beams and could discern flaws in either materials or fabrication. The tests would be rapid, inexpensive, and the cost of the equipment is reasonable. The testing would provide hardcopy documentation of the structural integrity of the beams. The cause of a weakened beam could be determined by other follow-up nondestructive and partially destructive tests.

RECOMMENDATIONS

The following recommendations are provided for consideration by Transportation Cabinet personnel.

1. The use of maturity meters would be beneficial if carefully incorporated into construction and fabrication shop practice. Consideration should be given to further research on the application of maturity meters and the use of rapid heating to replicate maturity meter readings from large concrete pours and from steam-cured beams.

2. Dynamic testing may be a promising method for nondestructive testing of shop precast beams. Consideration should be given to application of that method for shop inspections.

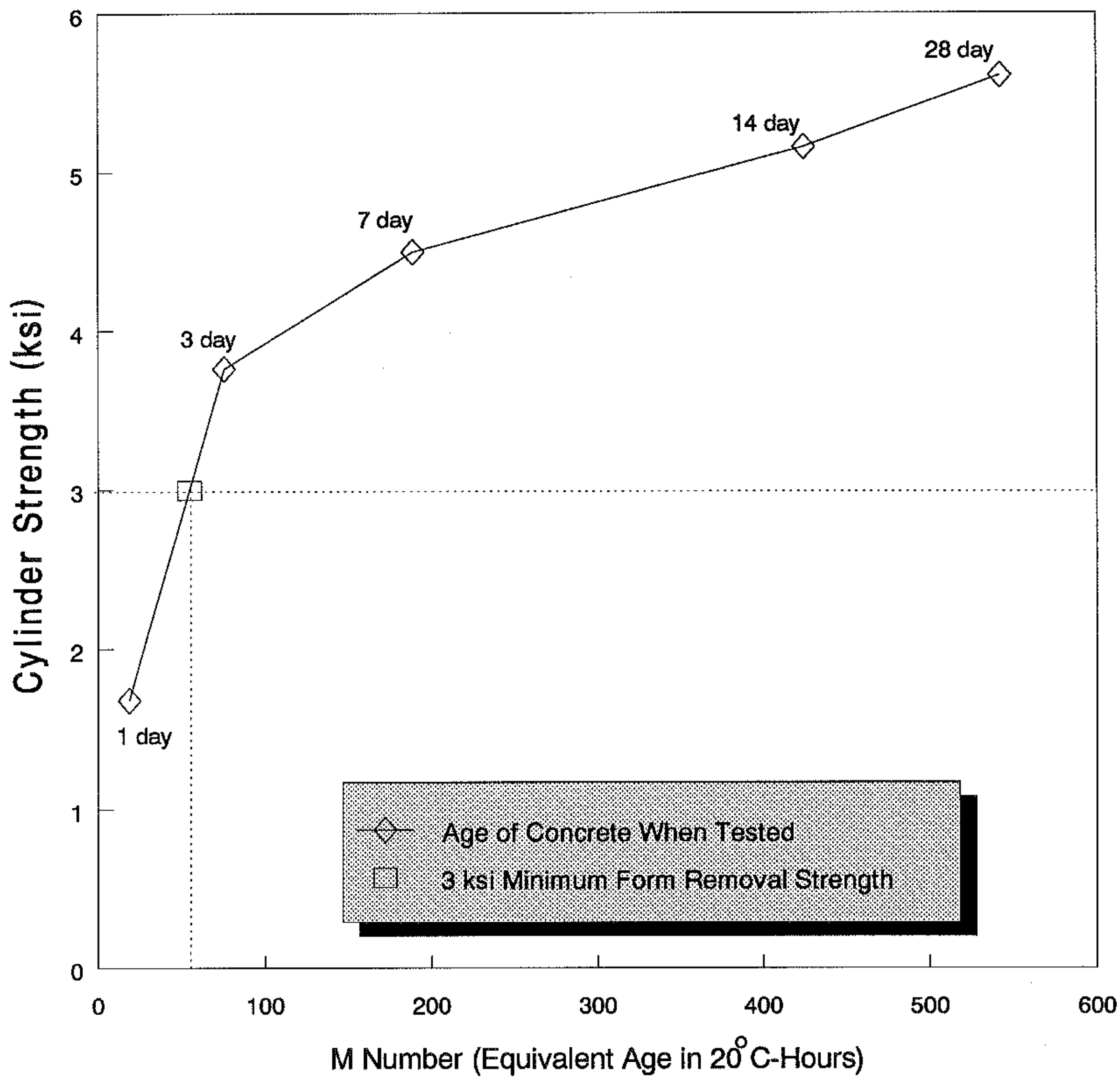


Figure 1. Average Cylinder Strengths for Class A Concrete Without Retarder vs Maturity Meter Readings (M Number)

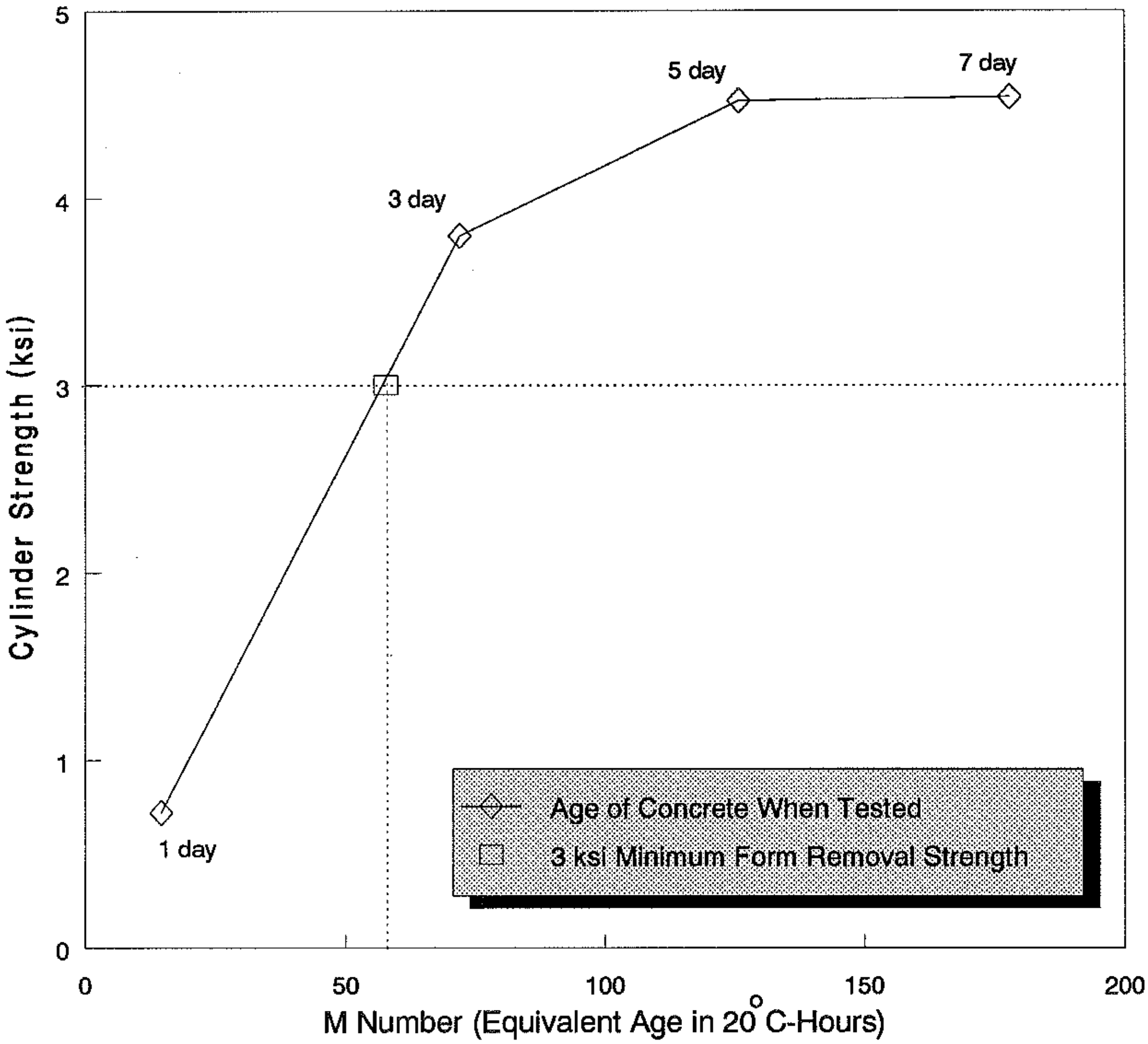


Figure 2. Average Cylinder Strengths for Laboratory Control Tests Using Class A Concrete with Retarder (for Pier Cap 1) vs Maturity Meter Readings (M Number)

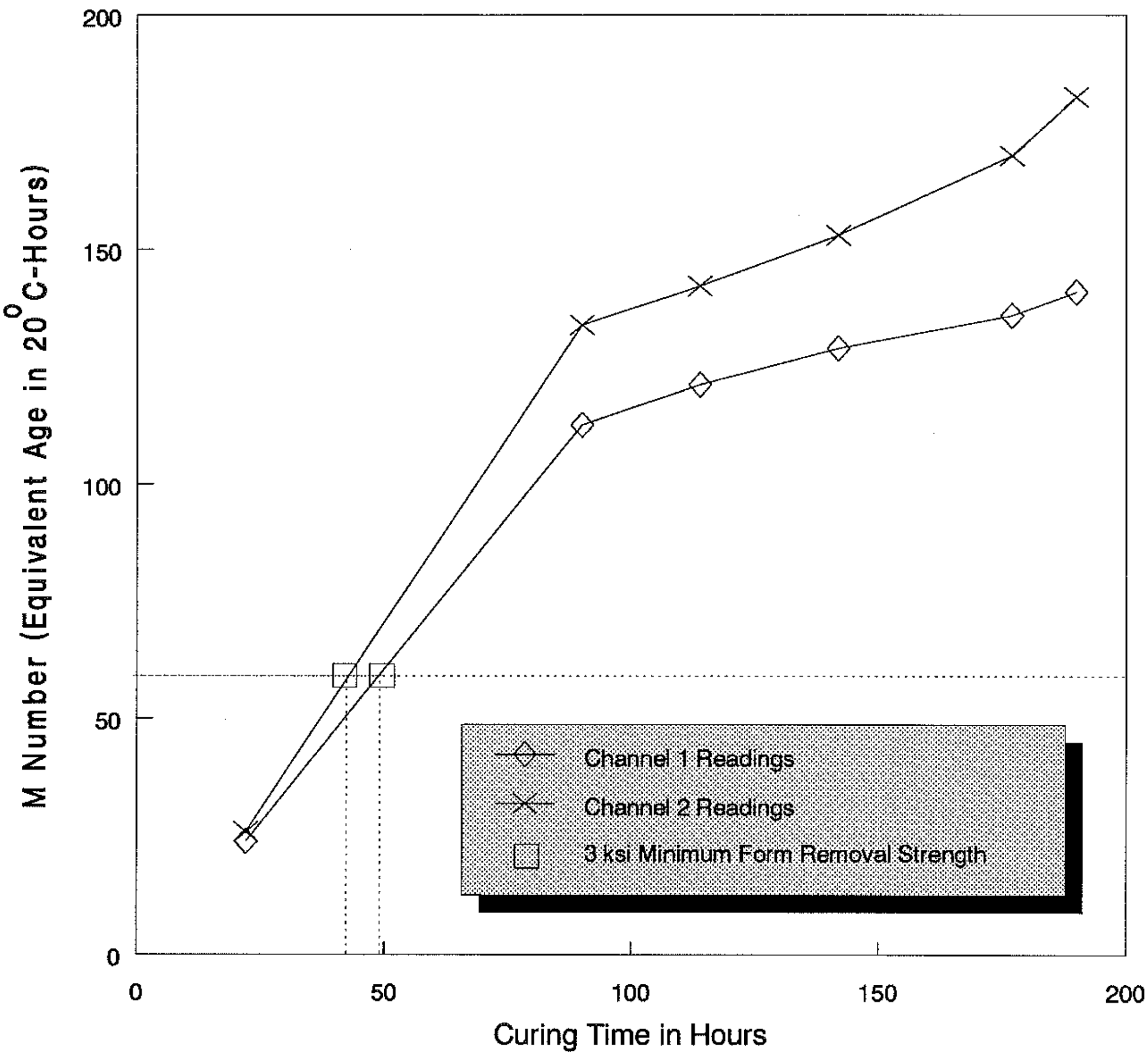


Figure 3. Maturity Meter Readings (M Numbers) vs Actual Cure Time for Class A Concrete with Retarder (for Pier Cap 1).

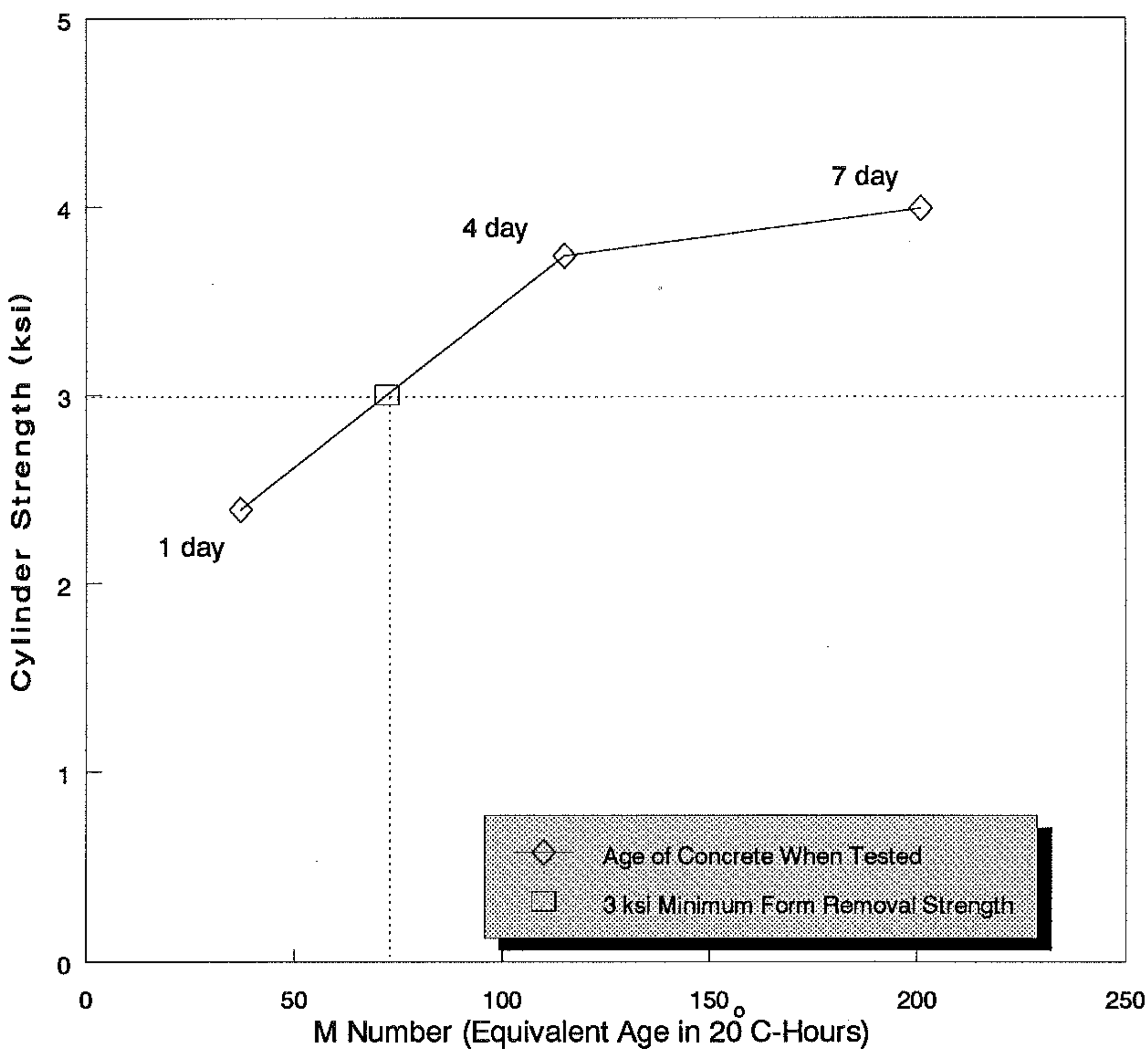
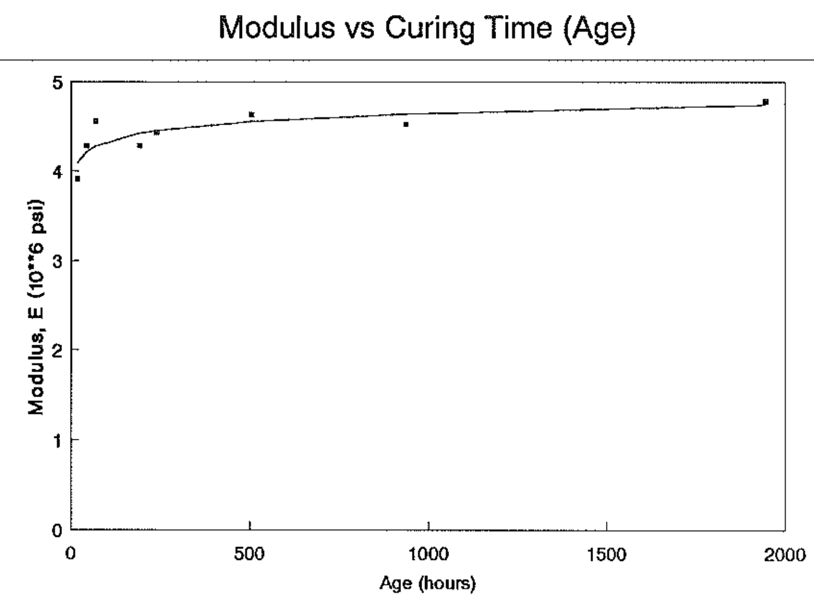


Figure 4. Average Cylinder Strengths for Laboratory Control Tests Using Class A Concrete With Retarder (for Pier Cap 2) vs Maturity Meter Readings (M Numbers)



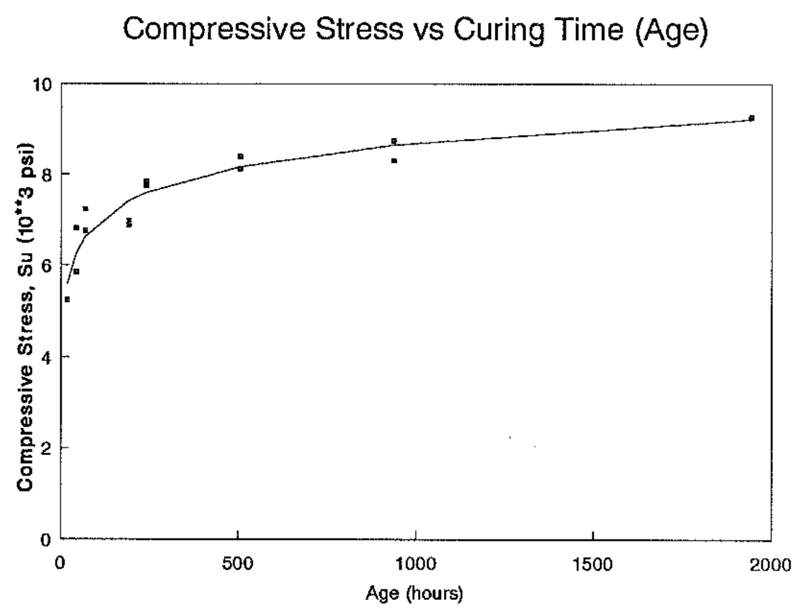
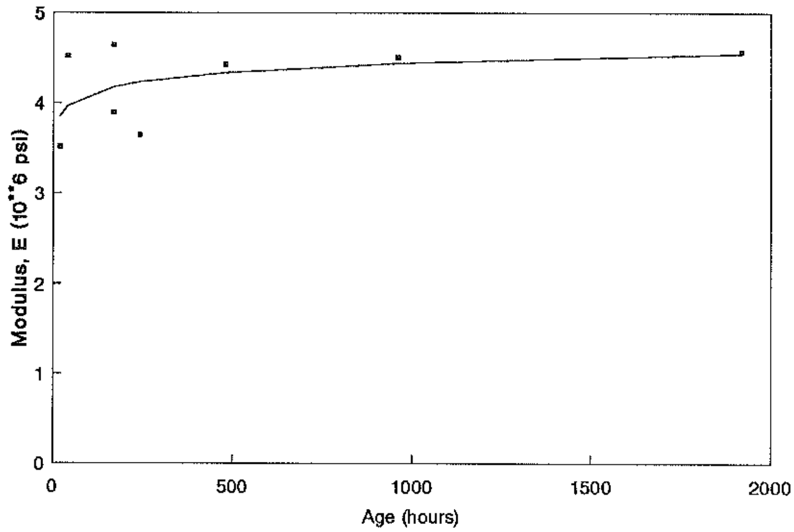


Figure 5. Modulus and Compressive Stress Data for Beam 305 Concrete Using Logarithmic Regression.

Modulus vs Curing Time (Age)



Compressive Stress vs Curing Time (Age)

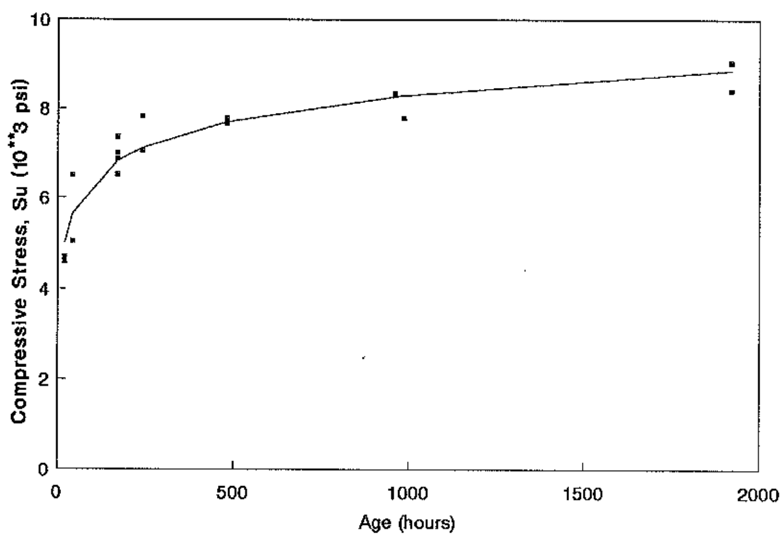
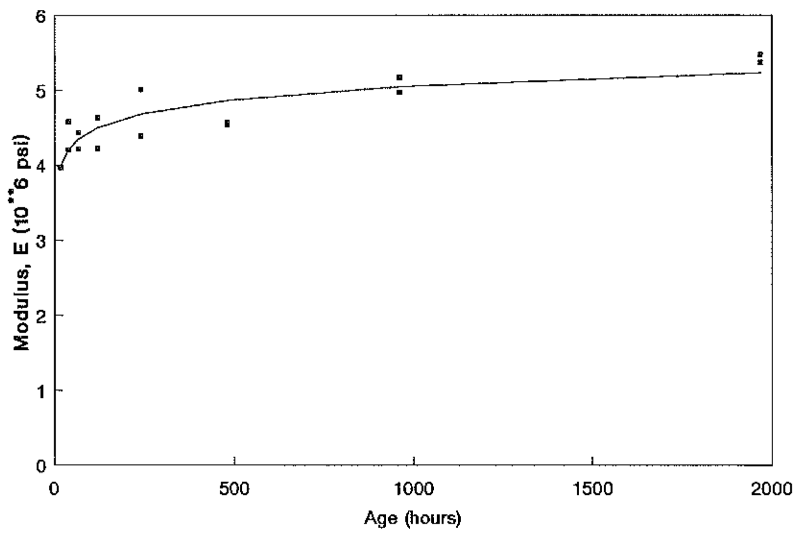


Figure 6. Modulus and Compressive Stress Data for Beam 306 Concrete Using Logarithmic Regression.

Modulus vs Curing Time (Age)



Compressive Stress vs Curing Time (Age)

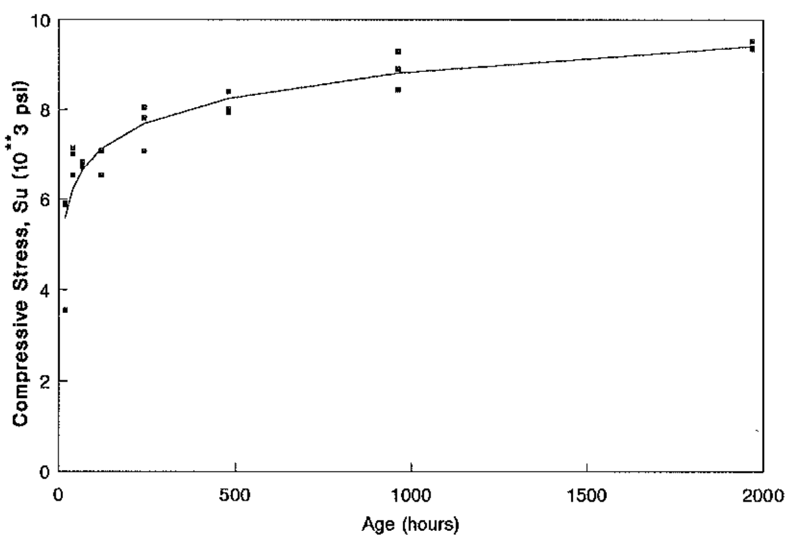
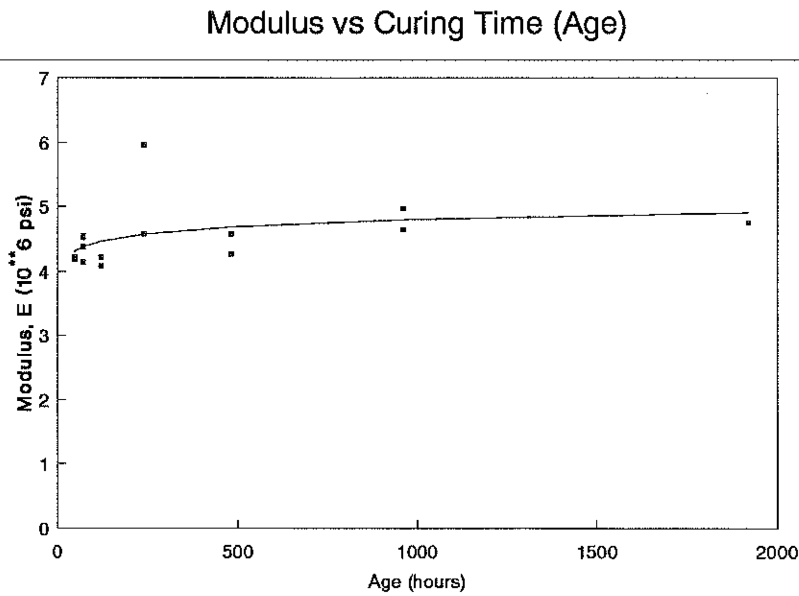


Figure 7. Modulus and Compressive Stress Data for Beam 405 Concrete Using Logarithmic Regression.



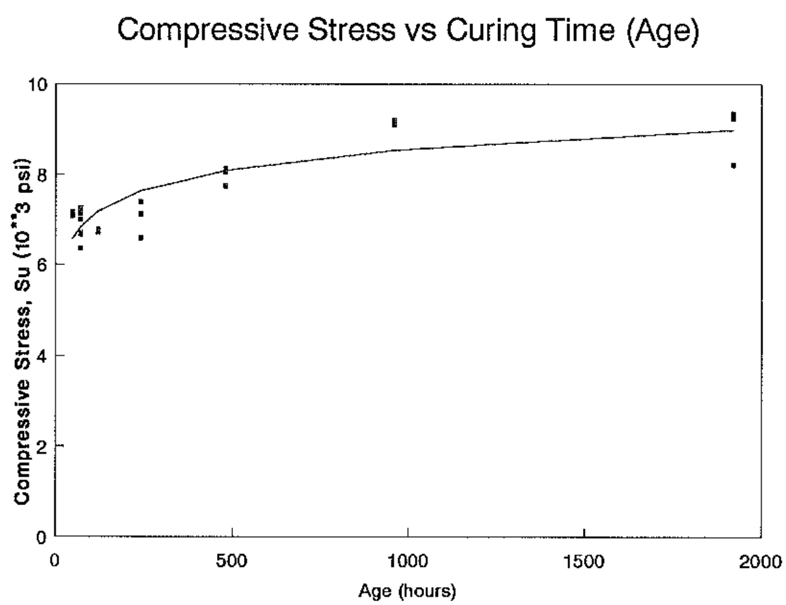
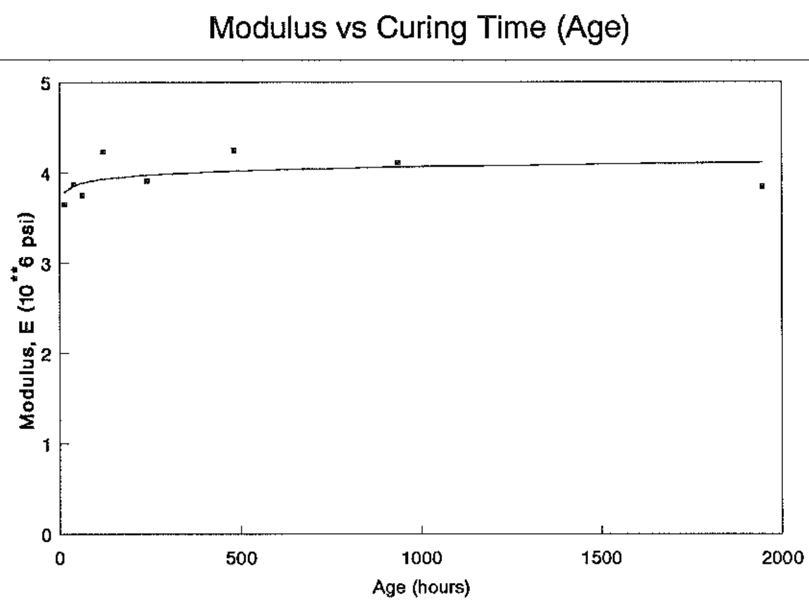


Figure 8. Modulus and Compressive Stress Data for Beam 406 Concrete Using Logarithmic Regression.



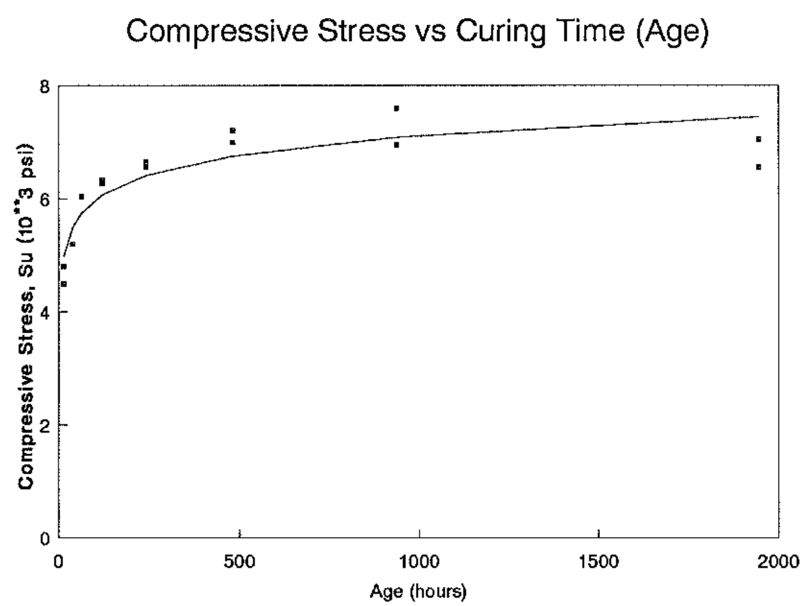
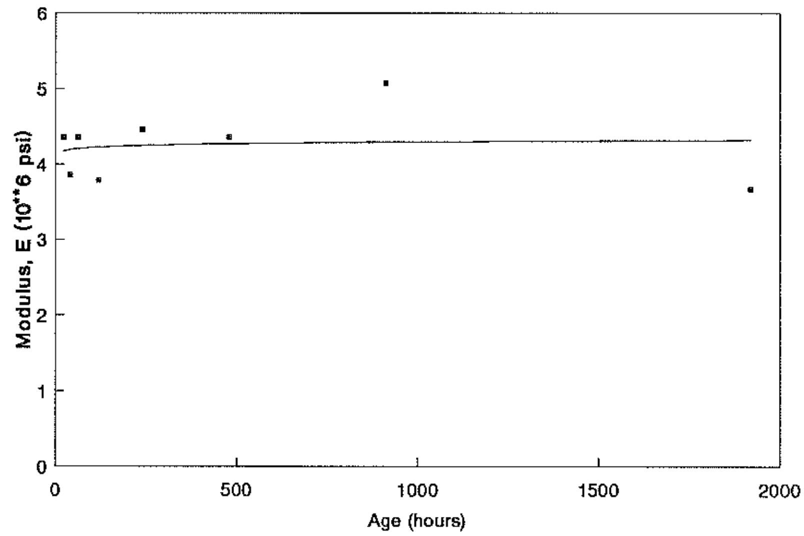


Figure 9. Modulus and Compressive Stress Data for Beam 505 Concrete Using Logarithmic Regression.

Modulus vs Curing Time (Age)



Compressive Stress vs Curing Time (Age)

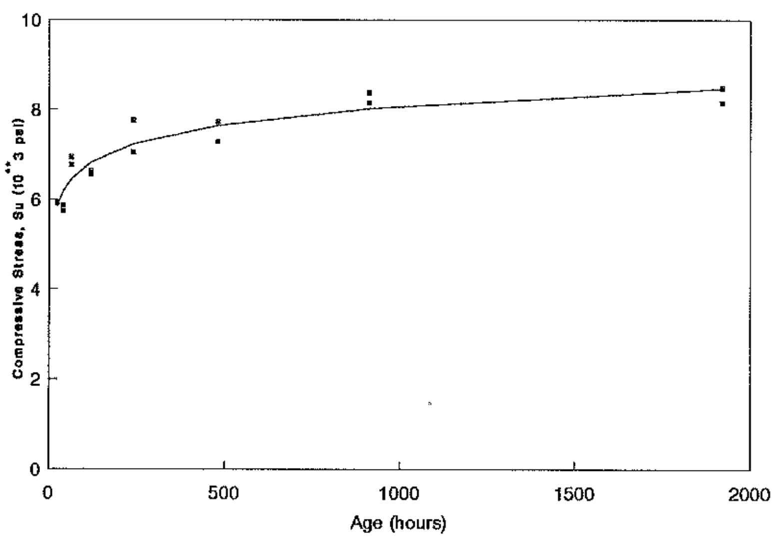
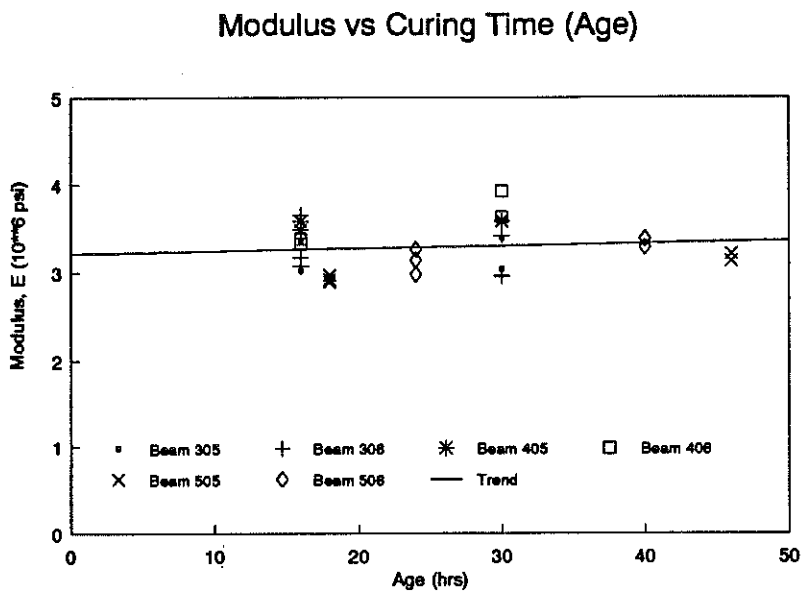


Figure 10. Modulus and Compressive Stress Data for Beam 506 Concrete Using Logarithmic Regression.



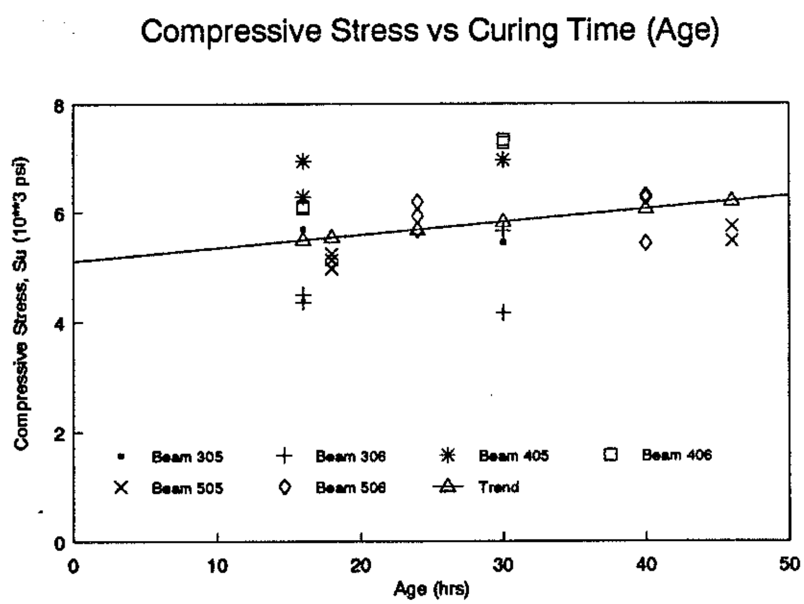


Figure 11. Modulus and Compressive Stress vs Curing Time (Age) of Concrete for All Beams Tested by KTC Personnel.

Table 1. KyTC Division of Materials
Modulus (E) and Compressive Stress (Su) Values
for Semi-Lightweight Concrete
Used in Beam 305 of the Twelvemile Bridge

Cyl. No.	E (10**6 psi)	Su (psi)	Age (hrs)
RB 82	3.91	5,240	18
RB 83		5,850	43
RB 84	3.95	6,810	43
RB 85		7,230	68
RB 86	3.80	6,760	68
RB 87		6,990	192
RB 88	3.91	6,880	192
RB 89		7,750	240
RB 90	4.13	7,850	240
RB 91		8,400	504
RB 92	4.31	8,130	504
RB 93		8,303	936
RB 94	4.42	8,750	936
RB 95		9,270	1,944
RB 96	4.48	9,270	1,944

Table 2. KyTC Division of Materials
Modulus (E) and Compressive Stress (Su) Values
for Semi-Lightweight Concrete
Used in Beam 306 of the Twelvemile Bridge

Cyl. No.	E (10**6 psi)	Su (psi)	Age (hrs)
RB 97		4,590	19
RB 98	3.16	4,690	19
RB 99		5,040	41
RB 100	4.00	6,500	41
RB 101		6,520	168
RB 102	4.21	7,000	168
RB 103		7,370	168
RB 104	3.58	6,870	168
RB 105		7,060	240
RB 106	3.42	7,830	240
RB 107		7,680	480
RB 108	4.10	7,780	480
RB 109		7,790	984
RB 110	4.19	8,350	960
RB 111		8,420	1,920
RB 112	4.57	9,040	1,920

Table 3. KyTC Division of Materials
Modulus (E) and Compressive Stress (Su) Values
for Semi-Lightweight Concrete
Used in Beam 405 of the Twelvemile Bridge

Cyl. No.	E (10**6 psi)	Su (psi)	Age (hrs)
RB 35		3,550	18
RB 33		5,910	18
RB 34	3.62	5,870	18
RB 36		6,540	39
RB 37	3.86	7,140	39
RB 38	4.58	7,000	39
RB 39		6,740	67
RB 41	3.88	6,840	67
RB 40	4.06	6,720	67
RB 42		7,060	120
RB 43	4.24	7,090	120
RB 44	3.99	6,530	120
RB 45		8,050	240
RB 46	4.61	7,820	240
RB 47	4.01	7,070	240
RB 48		8,400	480
RB 49	4.26	8,010	480
RB 50	4.24	7,930	480
RB 51		8,910	960
RB 52	4.61	8,450	960
RB 53	4.80	9,290	960
RB 54		9,520	1,968
RB 55	5.03	9,370	1,968
RB 56	4.98	9,340	1,968

Table 4. KyTC Division of Materials
Modulus (E) and Compressive Stress (Su) Values
for Semi-Lightweight Concrete
Used in Beam 406 of the Twelvemile Bridge

Cyl. No.	E (10**6 psi)	Su (psi)	Age (hrs)
RB 60		7,170	47
RB 61	3.93	7,130	47
RB 62	3.87	7,070	47
RB 57		7,250	70
RB 58	4.38	7,130	70
RB 59	4.14	7,000	70
RB 63		6,370	71
RB 64	4.12	6,710	71
RB 65	3.99	6,670	71
RB 66		6,750	120
RB 67	3.74	6,720	120
RB 68	3.90	6,780	120
RB 69		7,120	240
RB 70	3.64	7,390	240
RB 71	5.33	6,590	240
RB 72		7,750	480
RB 73	3.95	8,060	480
RB 74	4.21	8,130	480
RB 75		9,110	960
RB 76	4.34	9,110	960
RB 77	4.64	9,200	960
RB 78		9,350	1920
RB 79		8,220	1920
RB 80	4.44	9,240	1,920

Table 5. KyTC Division of Materials
~~Modulus (E) and Compressive Stress (Su) Values~~
for Semi-Lightweight Concrete
Used in Beam 505 of the Twelvemile Bridge

Cyl. No.	E (10**6 psi)	Su (psi)	Age (hrs)
RB 1		4,800	13
RB 2	3.31	4,490	13
RB 4	3.87	5,200	38
RB 6	3.75	6,040	62
RB 7		6,340	120
RB 8	3.83	6,280	120
RB 9		6,650	240
RB 10	3.57	6,560	240
RB 11		6,990	480
RB 12	3.93	7,200	480
RB 13		7,600	936
RB 14	3.56	6,950	936
RB 15		6,560	1,944
RB 16	3.55	7,050	1,944

Table 6. KyTC Division of Materials
Modulus (E) and Compressive Stress (Su) Values
for Semi-Lightweight Concrete
Used in Beam 506 of the Twelvemile Bridge

Cyl. No.	E (10**6 psi)	Su (psi)	Age (hrs)
RB 17		5,940	23
RB 18	3.90	5,920	23
RB 19		5,860	40
RB 20	3.50	5,730	40
RB 21		6,770	63
RB 22	3.98	6,936	63
RB 23		6,630	120
RB 24	3.57	6,560	120
RB 25		7,050	240
RB 26	4.13	7,770	240
RB 27		7,730	480
RB 28	4.00	7,290	480
RB 29		8,150	912
RB 30	4.36	8,380	912
RB 31		8,480	1,920
RB 32	3.46	8,150	1,920