Instrumentation and Evaluation of Prestressed Pavement Section in Pennsylvania

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During 1988 and 1989, a 3.2-km (2-mi) section of a prestressed pavement was constructed in Blair County, Pennsylvania as part of a new roadway along U.S. 220. In conjunction with the construction, the Pennsylvania Department of Transportation sponsored a study to instrument the prestressed pavement. Instrumentation was installed to monitor concrete strains, slab horizontal and vertical movements, and response of the pavement truck loading. This paper provides the details of the pavement instrumentation program and a summary of the data collected.

Most problems associated with conventional concrete pavements occur at joints and cracks. The most commonly observed distress in conventional concrete pavements are cracking, faulting, pumping, spalling, and joint sealant failures. In addition, continuously reinforced pavements may develop punchouts. These distress items in conventional concrete pavements contribute to reduced pavement serviceability, need for frequent maintenance, and traffic delays.

The use of prestressed (post-tensioned) concrete pavements has been suggested for many years to obtain extended service life and reduced maintenance. Conventional concrete pavements are designed on the basis of concrete's relatively low modulus of rupture (flexural tensile strength) without effectively utilizing the natural advantages of its high compressive strength. In prestressed pavements, precompression in the concrete because of prestressing allows for increased allowable stresses in the tensile and flexural state. This permits elimination of intermediate cracking because of both concrete volume changes and traffic loading, resulting in a decrease in the required number of transverse joints. Transverse joints in prestressed pavement may be spaced at 122 to 183 m.

Prestressed concrete pavements were first built over 40 years ago. There have been five major prestressed highway concrete pavements built in the United States, which have sufficient age to demonstrate the potential for improved serviceability and performance. A major study during 1985 sponsored by FHWA was conducted between 1976 and 1980 to develop design and construction procedures for prestressed concrete pavements (*1–3*). Work related to the Texas prestressed pavement project also included development of a thickness design procedure for prestress pavement overlays (*4–6*). The five U.S. projects are as follows:

- Virginia: Dulles International Airport Access Roadway, 1971;
- Pennsylvania: Near Harrisburg, 1973;
- Mississippi: U.S. 84, Brookhaven, 1975;
- Arizona: Superstition Freeway near Tempe, 1977; and
- Texas: I-35, Waco, 1985.

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During 1989, the Pennsylvania Department of Transportation (PennDOT) sponsored a study to instrument and evaluate the behavior of a prestressed pavement section built along a section of U.S. 220 near Altoona in Blair County, Pennsylvania. The instrumentation and monitoring plan developed was designed to provide information on slab horizontal movements, concrete strains (stresses), and response of the prestressed pavement under truck loading. Details of the instrumentation plan and monitoring of the Pennsylvania prestress pavement are provided in this paper. More comprehensive details of the study are given in the work of Okamoto et al. (*7*).

U.S. 220 PRESTRESSED PAVEMENT

The 3.2-km section of prestressed pavement was constructed during Fall and Winter of 1988 along a section of U.S. 220 in Blair County, Pennsylvania. The prestressed pavement was in the northbound lanes of the four-lane divided highway. The 26,260 m^2 of prestressed pavement is composed of 60 slabs 7.32 m wide and 178 mm thick. Active joints are spaced at 122 m. Slabs between the 122-m joints consist of 120.8 m of main slab and 1.2 m of gap slab. Tied concrete shoulders (using tie bolts) were incorporated with the traffic lanes. The outside tied concrete shoulder is 177.8 mm thick and 3.1 m wide. The inside tied concrete shoulder is 178 mm thick and 1.2 m wide. Shoulders are jointed plain concrete pavement with joints spaced at 6.1 m. Expansion joints were used at the 122-m spacing to match the expansion joints of the prestressed pavement. The cross-section of the prestressed pavement consisted of the following (from top down):

- 178-mm thick prestressed pavement;
- 2 layers of 0.15-mm (6-mil) polyethylene sheets;
- 127-mm thick lean concrete base course;
- 76-mm thick open-graded subbase;
- 76-mm thick Type 2A subbase; and
- Natural compacted subgrade.

Minimum compressive strength for Class AA concrete for the prestressed pavement was specified to be 20.7 MPa at 7 days and 25.8 MPa at 28 days. The following are the prestressing details for the U.S. 220 project:

- Plastic-encased seven-wire strand, grade 1860 MPa;
- Strand diameter: 15.2 mm;
- Strand spacing: 457 mm;
- Number of tendons: 15;
- Ultimate strength of strand: 261 kN;

- Stressing load (80 percent of ultimate): 209 kN; and
- Specified tendon placement: 13 mm below mid-depth.

The prestressed pavement was designed using the procedures given in the work of Nussbaum et al. (*1*) and Tayabji et al. (*2*). The minimum effective mid-slab prestress was computed to be about 400 kPa. Slab end movements were computed using the procedures given in the work of Nussbaum et al. (*1*) and Tayabji et al. (*2*). Total maximum slab end movement at a joint was computed to be 89 mm based on full slab length or 61 mm based on partial slab movement. The calculations based on full slab length are considered to be conservative, as the actual measured joint movements are typically less than those computed because only a smaller portion of the slab is involved in the movement.

Construction Details

Paving for the traffic lanes was performed in 7 days between November 11 and 18, 1988. Air temperatures during the paving period ranged from −1°C to 21°C and averaged approximately 4°C. Concrete temperatures after initial heat of hydration averaged approximately 7°C. Gap slabs were placed and stress transferred in January 1989. Tied jointed concrete shoulders were placed during July 1989.

Construction Specifications

This section presents a brief outline of applicable construction specifications that were unique to the prestressed pavement.

- Friction-reducing membrane: two layers of 0.15-mm (6 mil) thick polyethylene sheets.
- Temporary jacking bridge: the temporary bridge proposed by the contractor consisted of a 51-mm diameter nut internally threaded to accommodate threading of a 400-kN monostrand anchor-wedge system. The 51-mm diameter nut bears on a 13-mm thick plate. The temporary jacking face was used at the north end of the nominal 120.8-m long main slabs. A 1.2-m long gap was used to accommodate prestressing operations.
- Prestressing operations: prestressing operations were conducted as follows:

  1st Step—107 kN @ concrete strength of 6.9 MPa;
  2nd Step—variable; and
  3rd Step—323 kN

  Strands were tensioned from one slab end and then the other at about the same time. Elongation of each tendon was measured and compared to theoretical frictionless elongation.

- Gap slab construction: gap slabs were to be constructed approximately 30 days after main slab placement. Because of late fall paving in November 1988, the gap slabs were not placed until January 1989. The gap slabs were prestressed by transferring the prestress from the temporary jacking face of the main slabs to the active joint side face of the gap slabs. A section of the gap slab is indicated in Figure 1.
- Compression seals: specified seals were required to accommodate 44-mm movement at active joints. This design movement was established based on experience at the existing prestressed pavement near Harrisburg, Pennsylvania. The compression seal was 102 mm wide, 64 mm in height, and required a groove depth of 79 mm.
- Dowel caps: stainless steel dowel bars 38 mm in diameter by 457 mm long, spaced at 457 mm, were used at active joints. Dowel caps were used at one dowel end.
- Shoulders: tied concrete shoulders were constructed after gap slab construction. Because of late fall paving, the shoulders were not

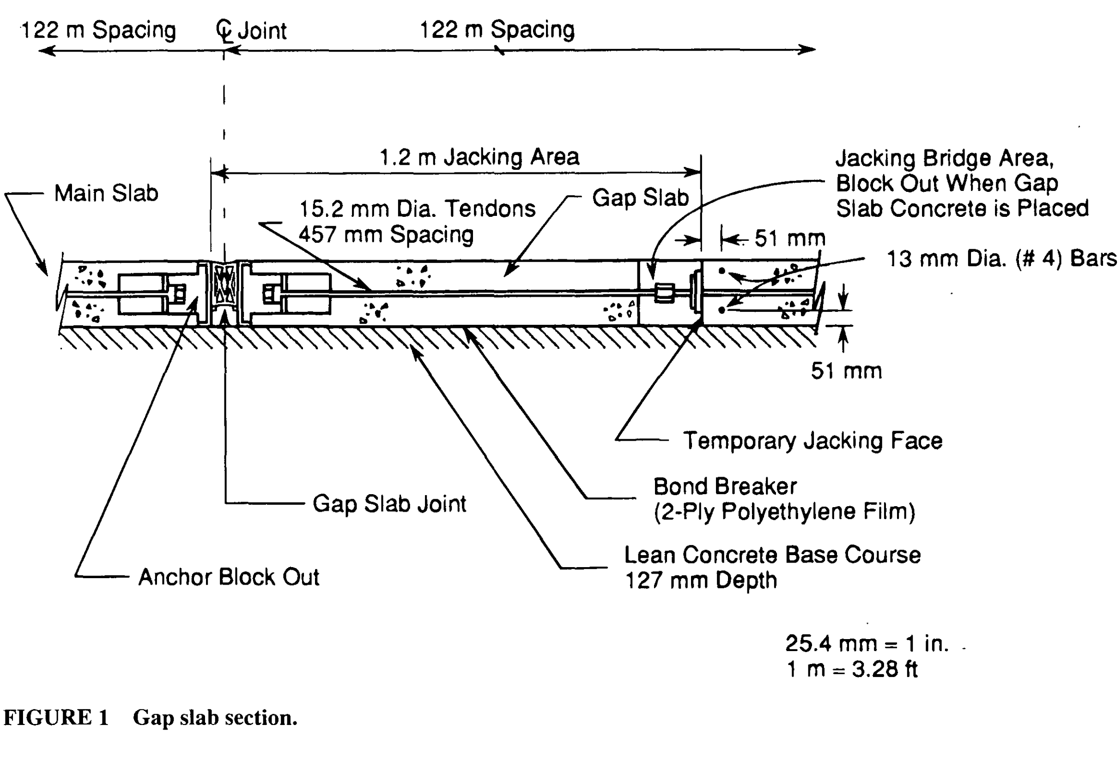

FIGURE 1 Gap slab section.

placed until July 1989. The shoulders were placed over the 127-mm thick lean concrete base used under the mainline pavement. Two applications of white pigmented wax-based curing compound were used over the lean concrete base to reduce slab-to-base friction.

INSTRUMENTATION PLAN FOR THE U.S. 220 PRESTRESSED PAVEMENT

Short-Term Monitoring

The objective of the short-term monitoring plan was to obtain information related to slab movements and concrete strains and stresses during the early age (over a period of several days) of the pavement. Pavement monitoring was initiated soon after concrete placement and was continued for up to 15 days. Shoulders and gap slabs were not yet constructed during the short-term monitoring period. Instrumented Slab 1 cast on November 14, 1988 was the 10th two-lane slab paved. Instrumented Slab 2 cast on November 17, 1988 was the 22nd slab of 30 slabs (two-slab pair) paved. Gap slabs were placed approximately 50 days after construction.

Long-Term Monitoring

The objective of the long-term monitoring plan was to obtain information related to slab movements and concrete strains after concrete shoulder placement. Long-term monitoring of the installed instrumentation and condition survey was conducted as follows:

1. During April 1989 (early spring time), the initial post-construction cracking survey was performed (before shoulder placement).
2. At the time of the August 1989 load testing, measurement of joint widths and slab length changes, monitoring of installed instrumentation (concrete strains, temperature), and second post-construction cracking survey (after shoulder placement) were performed.
3. During March 1990 (late winter), measurement of joint widths and slab length changes and third post-construction cracking survey were performed.

Tied concrete shoulder construction was performed in July 1989, before load testing. Load testing was conducted after the construction of concrete shoulders during the first week of August 1989. Load testing was conducted over a period of 2 days. The falling weight deflectometer (FWD) testing was done in September 1989.

Load Testing

The objective of pavement load testing was to determine the response (strain and deflection) of the prestressed pavement to actual single-axle and tandem-axle truck loadings. In addition, a FWD was used to characterize the deflection response of all 30 outside lane slabs.

Instrumentation Layout

Two inside lane slabs were monitored both during the short- and long-term visits. Joint widths at all 31 joints were measured during the long-term monitoring phase of the study. The two slabs monitored were selected from the third and sixth day of paving. Similar to the prestressed slabs, the second instrumented slab had two layers of polyethylene placed under the jointed concrete shoulders. The instrumentation was placed at two sites (Sites 1 and 2) at each of five stations (Stations A, B, C, D, and E), as indicated in Figure 2. At Sites 1 and 2, the embedded sensors (strain gauges and thermocouples) were installed by boxing out concrete at these locations, removing the boxed-out concrete, installing the sensors mounted on "chairs," and carefully replacing the concrete in the box-out area around the sensors. Sensors were prewired and positioned under the boxes before paving. Wiring for the instrumentation was run through a polyvinyl chloride conduit buried 102 mm into the granular subbase. The conduit was prepositioned before lean concrete construction. The lean concrete base was later cored to provide access to the conduit. A discussion of the specific instrumentation used in the U.S. 220 project is given in the work of Okamoto et al. (*7*).

Load Testing Details

Full-scale load testing of the prestressed pavement was conducted soon after the concrete shoulders were constructed and before the pavement was opened to traffic. Load testing was conducted at the two instrumented slabs. At each slab, instrumentation was installed to determine mid-slab edge strain (outside lane), mid-slab deflection, joint deflection, and joint load transfer. Load tests were also conducted on an adjacent new 279-mm thick jointed plain concrete pavement (referred to as Slab 3). This pavement was placed over a 102-mm open-graded base and 127-mm granular subbase, and has transverse-skewed doweled joints placed at 6.1 m. Strain and deflection data were recorded for 8,080-kg single-axle and 17,750-kg tandem-axle truck loadings. Data were recorded for axle placements along the outside lane edge and a distance of 457 mm inward from the lane edge. Trucks were operated at creep speed to measure load-induced deflection and strain. Load testing was carried out several times during the day, starting early in the morning, to determine the effect of temperature variations on slab deflections and concrete strains because of loading.

In conjunction with the full-scale load testing, FWD testing was conducted along all 30 outside lane slabs. Testing for each slab was conducted at joints and interior stations along the outside lane.

Materials Testing

Data on concrete tests performed by others were obtained for analysis of early age strength gain (post-tensioning operations). Compressive strength testing was specified by PennDOT. Concrete cylinder strengths at early age were tested for initial and final post-tensioning considerations. Modulus of elasticity and compressive strength tests were also performed on cylinders fabricated during concrete placement. Testing was conducted to establish a project-specific relationship to predict modulus of elasticity from compressive strength data.

SUMMARY OF SHORT-TERM MEASUREMENT DATA

Ambient and Concrete Temperatures

Ambient air and concrete temperatures were recorded every hour for the first 17 days for instrumented Slab 1 and 14 days for instru-

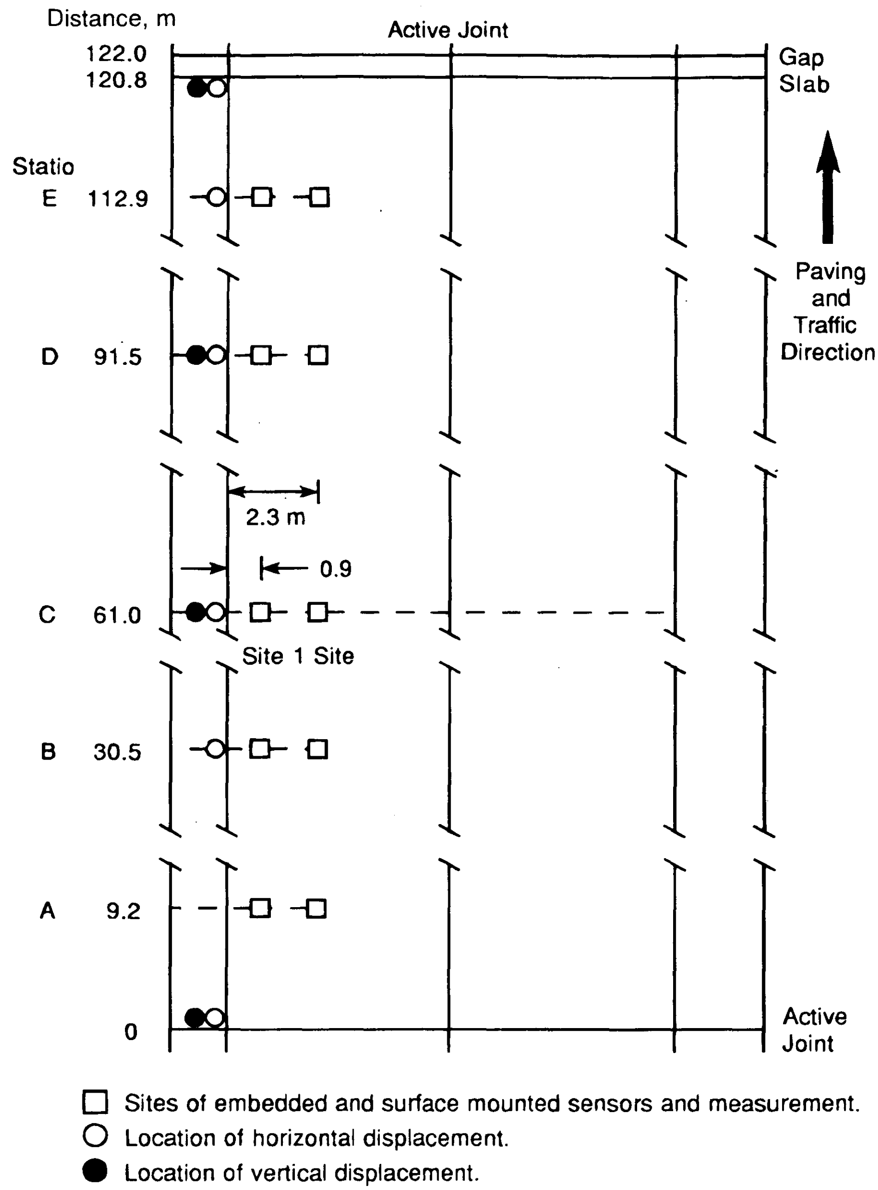

FIGURE 2 Instrumentation layout at a slab.

mented Slab 2. Air temperatures at the site during this phase of testing typically ranged from −1°C to 10°C. Typically, slab temperatures decreased from 21°C to 4°C within 5 days. The use of straw insulation sandwiched between two layers of white polypropylene retained the heat of hydration. The typical daily range in concrete temperatures was 3°C. No significant temperature fluctuation was observed, even when the insulating layers were removed, because of relatively cool and constant ambient temperatures and minimal solar radiation effects.

Concrete temperature 152 mm from the surface was slightly higher than the temperature at 13 mm from early evening to about mid-day (12:00 noon). The temperature differential between the 13-mm and 152-mm depths was minimal over the entire monitoring period. Typically, for both instrumented slabs, the differential temperature was less than 1°C during the short-term monitoring period.

Concrete Strain

The concrete strain was monitored with embedment, sister (strain gauges bonded to deformed bars), and Carlson gauges positioned at slab mid-depth. Embedment gauges were also positioned at 38- and 140-mm depths to provide data on strain variation with depth. Surface-mounted strain gauge data were also used to monitor concrete strain changes. Initially, it was planned to mount the gauges on the surface approximately 24 to 36 h after concrete placement, after the concrete had attained sufficient strength. Because of delays in starting construction, the concrete was placed in much colder than anticipated weather. The relatively slower concrete strength gain and retention of moisture by polypropylene and straw insulation prevented gauges from being adequately bonded to the surface until after about 60 h. Strain changes were similar in trend to near-surface embedment gauges.

The measured strains include effects of temperature, subbase friction, prestress, concrete creep, and drying shrinkage. Concrete creep and drying shrinkage effects increase strain (compression-positive sign convention) as the slab shortens. Since average daily concrete temperatures remained uniform between the second and third prestress levels, the general strain increase is attributed to shrinkage and creep.

Prestress Stages

For both Slabs 1 and 2, the strains dropped significantly (loss in prestress) after the first prestress stage at 1.7 and 1.4 days, respectively. This drop was fully recovered with the second prestress stage approximately 24 h later. The loss can be attributed to concrete contraction caused by shrinkage and temperature. The concrete temperature record during the first day for both slabs indicated a downward trend as the slab heat of hydration dissipated. During the first 36 h for Slab 2, the measured concrete temperatures approached 21°C. By the second stage of prestress, concrete temperatures (average of four depths) dropped to approximately 16°C. The effects of first-stage prestressing (post-tensioning) are clearly indicated to offset the tensile subbase restraint stresses built up as the slab cooled after the peak heat of hydration. Without the critical first stage of prestress at about 36 to 48 h, the tensile restraint stresses could have exceeded the concrete tensile strength.

Also, the subbase restraint had an effect on effective prestress. At slab mid-span (Station C), the prestress is significantly lower than near the ends at Stations D and B. The largest prestress was near the slab end where frictional subbase restraint was lowest.

Larger decreases in strain (drop in prestress) between the first and second prestress stages were measured at Station C for Slab 2. Decreases in sister gauge readings at Slab 2 Stations C and D after first-stage stressing possibly indicate that the gauges were not properly monitoring strain at 24 h. The change in strain at other stress levels indicated consistent increases. It appears that the gauges had not quite stabilized at 24 h for Slab 2.

Stresses were computed from strains and the modulus of elasticity estimated from compressive strength. Concrete stresses and strains generally increased more (compression) with each prestress level at the ends than near mid-span. At the final prestress stage, the stress near the slab ends for the three gauge types ranged from 1.0 to 1.5 MPa higher than that at mid-span for Slab 1. For Slab 2, the stresses at the ends ranged from 0.4 to 2.2 MPa higher than at mid-span. Generally for Slab 1, both sister and embedment gauges indicated the final prestress levels were somewhat symmetrical around mid-span. For Slab 2 gauges, the final prestress was not as symmetrical. Final prestress at Slab 1 mid-span ranged from 0.6 to 0.7 MPa and averaged 0.7 MPa for the three gauge types.

The embedment gauge strain was monitored at depths of 38, 89, and 140 mm from the slab surface. The average stress difference for the first two prestress stages between top and bottom gauges was 124 and 41 kPa for Slabs 1 and 2, respectively. For prestress Level 3, the change in strain was generally higher at the top than at the bottom of the slab. The average third level prestress gradient was 227 and 138 kPa higher compressive stress at the top than at the bottom for Slabs 1 and 2, respectively. Final prestress gradient ranged from 427 kPa lower to 338 kPa higher compressive stress near slab bottom than the top. The overall average for both slabs at all stations (compressive stress) was about 21 kPa higher at slab top than at bottom.

Daily Strain Variations

Strains measured at mid-depth with Carlson gauges are indicated in Figure 3 for Slab 2. There is a decrease in compressive strain from early morning to mid-day as the slabs expand because of an increase in air temperature. As the slab temperature drops near mid-day, the compression strains increase. The apparent strain because of creep and drying shrinkage shortening of the slab are observed by the slowly rising general trend in strain.

Horizontal Slab Movement

Horizontal slab movement was monitored using dial gauges mounted on reference rods embedded 0.6 m into the subgrade. Monitoring was performed for both slabs three times a day (at approximately 8:00, 12:00, and 16:00) for about the first 2 weeks after concrete placement. Horizontal movements were monitored to evaluate the effects of post-tensioning, temperature variations, and initial concrete shrinkage. Movements were monitored at slab ends, mid-slab, and quarter points along the slab length.

The slab temperatures and gradient with depth did not vary because of the relatively cold air temperatures, low solar radiation, and use of straw insulation at earlier ages. Very small displacement magnitudes were measured throughout the day. Longitudinal end movements for Slab 1 at 17 days and Slab 2 at 14 days averaged 14 mm and 15 mm. Movements at 31 m from transverse joints averaged 8 mm.

Vertical Slab Movement

Vertical slab movements were monitored simultaneously with longitudinal displacement. Gauges were mounted on reference rods positioned 0.6 m into the subgrade. Curling because of temperature variations and prestressing was monitored at mid-slab and at both slab end corners. Similar to longitudinal displacement data, curling effects because of slab thermal gradients were negligible throughout the first day. Very small changes in curl were measured at the mid-span edge and corners as the slab temperature increased.

Curl, relative to the day of placement, was significantly higher at 4 days for Slab 1 at both the edges and corners. Similar increases were noted at 2 days for Slab 2. For both slabs, the curl magnitude within 1 or 2 days after casting was higher at the corners than at the mid-slab edges. The measured curl was about 1 mm at slab corners after the first few days. Typical of most days during the short-term monitoring period, temperature gradients ranged from 0.2°C/in. (temperature at bottom greater than top) to zero. This was partly because of the cool water-like temperatures and the straw cover used over the new pavement.

Tendon Elongations

Tendon elongation for all slabs was measured for each level of stressing. For the 120-m main slab prestressing, the total theoretical tendon elongation after final post-tensioning was 907 mm. Data provided by PennDOT indicated that measured elongations were generally less than the calculated elongations. Differences are mainly attributable to plastic casing friction and tendon wobble. Measured tendon elongations ranged from about 740 to 910 mm.

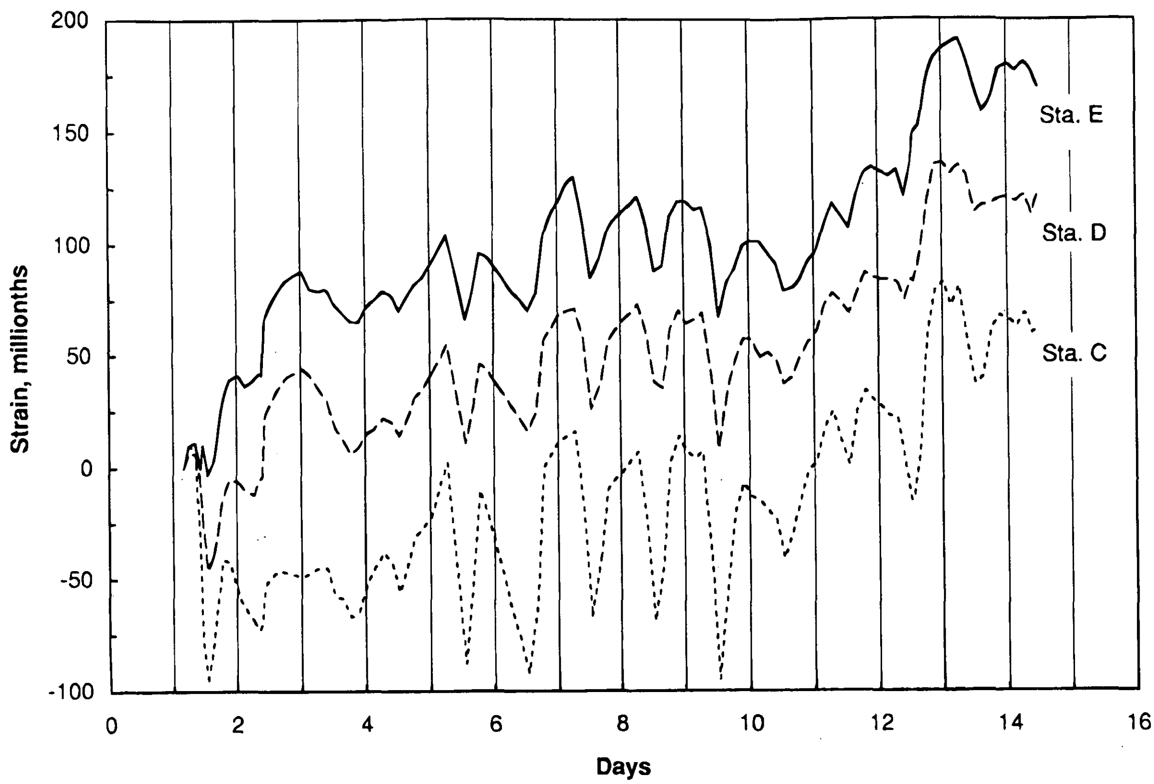

FIGURE 3 Slab 2 Carlson gauge strains.

SUMMARY OF LONG-TERM MEASUREMENT DATA

The objective of long-term monitoring was to obtain information related to slab movements and concrete strains and stresses after a period of several months. The monitoring of strains was conducted during the first week in August 1989 in conjunction with load testing.

Concrete Strain

Analysis of the sister and embedment gauge strain data relative to the initial short-term monitoring resulted in inconsistent trends and magnitudes. Concrete strain was monitored with the embedment, sister, and Carlson gauges. The embedment and sister gauges used in the short-term monitoring possibly drifted over the extended period of time. Since the sister and embedment gauge data could not be directly related back to the initial readings, the data was referenced to the summer visit initial readings.

The Carlson gauges used were initially selected during the project planning since they are considered very reliable over a long period of time. Because of the relatively high gauge cost, the number of gauges was limited to three per slab. The Carlson gauges, as expected, provided consistent strain data relative to the initial short-term monitoring approximately 9 months earlier. The Carlson gauge data reported is relative to the 17-h baseline used in the analysis of short-term strain monitoring. No strain data adjustments were made in Carlson gauge data to account for early and long-term creep and drying shrinkage effects.

Surface-mounted strain gauge data was also collected for Slab 2. Changes in strain with temperature were significantly larger compared to other gauges. Larger changes were attributed to effects of temperature on the gauge instead of changes in true concrete strain. Since the gauge data could not be reliably compensated for temperature, and the trends in strain were similar to embedment gauges at 38 mm below the surface, strain gauge data from surface-mounted gauges are not discussed in the analysis of long-term data.

Relative Strain Response

The slab response to thermal changes appears to be symmetrical. Changes in strain because of temperature at Stations A (9.2 m) and E (112.9 m) follow the same trend and magnitude change.

For Slab 1, embedment gauges at depths of 38, 89, and 140 mm were monitored at all five stations to evaluate the effects of temperature gradients on prestress distributions with depth. The gauges at depths of 38 mm generally indicated a larger strain-change magnitude than the gauges at depths of 89 mm. The gauge at mid-depth also consistently had a larger relative strain change than the gauges at 140 mm. This corresponds to the relatively larger temperature ranges recorded during peak daylight hours near the surface.

Absolute Strain Response

Carlson gauge data measured over a period of 24 h are relative to the initial short-term monitoring period (baseline 17-h age). No adjustments for early and long-term drying shrinkage or creep were made on the data. Because of long- and short-term creep and drying shrinkage effects, the slab shortening caused an increase in compressive strain. With slab shortening there is a loss in effective prestress.

Ranges in daily strain measured using Carlson gauges for both slabs was 92 to 144 $\times 10^{-6}$ mm/mm. No significant change in range was measured between the stations for Slab 2. For Slab 1, the range in strain was significantly higher at Station C than at Stations A and B. Measured strain magnitudes for Slab 1 ranged from 175 to 354 $\times 10^{-6}$ mm/mm at the three stations (relative to hour 17). For Slab 2, strain magnitudes ranged from 274 to 555 $\times 10^{-6}$ mm/mm at the three stations.

Joint Width

Joint widths were measured at each of the 30 transverse joints and at the two terminal joints using a measuring scale. Measurements were conducted at several different times of the day to evaluate opening and closing rates and magnitudes. Readings were taken 610 mm from the slab edges. Initially, both inside and outside lane joint measurements were taken. After several time periods no significant differences in joint width for the inside and outside lane movement were recorded.

Joint widths were measured August 3 and 4, 1989 and March 7 and 8, 1990. Air temperatures during the summer monitoring period ranged from 23°C to 33°C. Air temperature at the winter visit ranged from −8°C to 13°C. Daily transverse joint width changes on average were about 8 and 13 mm during the summer and winter visits, respectively. Larger changes were measured in the winter because of larger air temperature ranges. Overall average width for transverse joints was 44 and 73 mm for summer and winter monitoring, respectively.

Vertical Slab Movement

Vertical curl changes were measured with dial gauges mounted on reference rods positioned 1.5 m into the subgrade. Relative changes in curl were monitored at the slab corner, mid-span, and at other interior stations for both instrumented slabs. In addition to the prestressed slab curl, vertical movement was also recorded at a jointed concrete slab adjacent to the project section. Curl was measured in conjunction with full-scale load testing. Curl was measured for the outside (truck) lane.

No significant differences in magnitude or trend were measured at different stations of the prestressed slabs. Curl was similar at the corner, mid-slab, and intermediate stations for the prestressed slabs. Maximum curl deflection relative to initial readings was 0.5 and 0.7 mm for the 178-mm thick prestressed Slabs 1 and 2, respectively. Minimum curl (down) occurred at 12:30 p.m. and 1:00 p.m. for Slabs 1 and 2, respectively. Maximum relative curl for Slab 3, the 279-mm thick plain-jointed slab, was 0.5 mm.

LOAD TESTING DATA SUMMARY

Full-scale load testing was performed on the prestressed slabs during August 1989. Surface edge strain and deflection data for the two prestressed slabs and the one plain slab were measured for an 8,080-kg single-axle dual wheel and 17,750-kg tandem-axle dual wheel truck loading. Data under moving loads were recorded for wheel placement along the outside edges of the driving lane and at a distance of 457 mm inward from the edge. Trucks were operated at creep speed of 3.2 to 4.8 km per hour. FWD testing along all 30 outside lane slabs was conducted in September 1989.

Load Truck Response

A summary of measured load-induced edge strains (in millionths) is given below for mid-slab locations:

	Slab 1	*Slab 2*	*Slab 3*
Single-axle loading—morning			
edge loading	33	27	10
load at 457 mm from edge	14	8	2
Single-axle loading—afternoon			
edge loading	18	14	7
load at 457 mm from edge	8	9	3
Tandem-axle loading—morning			
edge loading	31	32	8
load at 457 mm from edge	22	—	3
Tandem axle loading—afternoon			
edge loading	10	21	7
load at 457 mm from edge	6	9	5

Average edge strains decreased significantly as the load shifted from edge to 457 mm from the edge. Load-induced strain in the prestressed slabs decreased during the afternoon as slabs curled down with temperature. Load-induced strain on the 279-mm thick plain concrete slab was less sensitive to curling effects. Thinner slabs have greater temperature gradients and therefore greater curl deformation than thicker slabs.

A summary of measured load-induced edge deflections (in mm) is given below for mid-slab locations:

	Slab 2	*Slab 3*
Tandem-axle loading along edge—morning	0.38	0.13
Tandem-axle loading along edge—afternoon	0.15	—

Measured deflections were sensitive to stationing for the prestressed slabs. Significantly higher deflections were measured at mid-span in the prestressed slabs than at stations away from mid-slab and at the joints. Differences in deflections with stationing were significant only for the earliest set of readings. For the last two sets of deflection data for each slab, when slabs had the least upward curl, there was no significant difference in deflection with stationing.

FWD Test Results

A Dynatest 8002 FWD was used to evaluate deflection load transfer efficiency and estimate modulus of subgrade reaction. Two types of testing were conducted. Joint testing was performed to calculate joint efficiency at each joint in the center of the outside (truck) lane. Basin deflection testing was done to calculate modulus of subgrade reaction.

The effective modulus of subgrade reaction was backcalculated for each slab at 61.0 m and 91.5 m. Load deflection transfer efficiency is the unloaded divided by loaded slab deflection (percent). A summary of joint efficiency and modulus of subgrade reaction data is given below:

	Subgrade Support, MPa/m	*Joint Effciency, percent*
Minimum	43	59
Maximum	103	94
Average	77	77
Standard deviation	14	9

CONDITION SURVEY DATA

The first condition survey of the 60 prestressed slabs was made in April 1989. Gap slabs were placed by the April 1989 visit and prestress transferred from the main slab to the gap slabs. Shoulders had not yet been placed. The second condition survey was performed approximately 2 weeks after tied shoulder construction during the long-term monitoring and load testing in August 1989. The third cracking survey was performed in March 1990 during the winter site visit to measure joint movements.

Tied concrete shoulders were added to the prestressed slabs in July 1989. The full-depth concrete shoulders were jointed plain concrete with 6.1-m joint spacing. Construction traffic was allowed on the prestressed slabs. Several hairline transverse cracks in the prestressed slabs were noted during the August 1989 condition survey. Cracking generally was within 6.1 m of the mid-span of prestressed slabs. Cracks were observed in the 5th through 11th and 15th through 18th two-lane slab sections. Two other slab sections, the 1st and 28th slabs, also had transverse cracking. All slabs with multiple cracks were from the third paving day. Of the five pairs of slabs cast on the third paving day, two had one crack, one had two cracks, and two had three cracks.

A follow-up survey in March of 1990 (second winter) indicated a significant amount of additional cracking. For the 30 pairs of prestressed slabs, 132 cracks were observed. Cracks were noted in all but 7 of the 30 pairs cast. Cracks were generally within the mid 61 m of the 122-m long prestressed slab. Eighteen pairs had five or fewer cracks. Four and five pairs of prestressed slabs had 7 and 9 cracks, respectively. The other three pairs of prestressed slabs had 11, 13, and 15 cracks at the time of the March 1990 survey. Cracks were noted in slabs cast on all paving dates with the exception of two slabs cast on the last paving day. Minor spalling was observed along several cracks in the prestressed slabs.

Similar to the prestressed slabs, the second instrumented slab had two layers of polyethylene placed under the jointed concrete shoulders. At the time of the March 1990 survey, the first instrumented slab (slab section 10 from the third paving day) had seven transverse hairline cracks. Instrumented Slab 2 (slab section 22 from the sixth paving day) did not have any cracks at the time of the March 1990 condition survey. Results of the August 1989 (first summer) and March 1990 (second winter) cracking survey are indicated in Figure 4.

The excessive number of cracks within slab sections 4 through 10 and 13 through 16 indicates that prestressing of the slabs was not effective. Cracking can be attributed to both construction problems and design deficiencies. Other problems associated with the tied concrete shoulder were the effects of differences in joint movements. Numerous shoulder slabs (outside lane) at the time of the March 1990 condition survey exhibited joint spalling and corner breaks. The prestressed slab expands and contracts over its full length. The jointed shoulder slabs only move over their 6.1-m lengths. The differential movement problem was clearly observed at prestressed slab active joints. The prestressed slab differential movement resulted in an observable offset in joint positions. At the time of the August 1989 survey, the prestressed slab joints were up to 25 mm away from the adjacent shoulder joint. The offset also

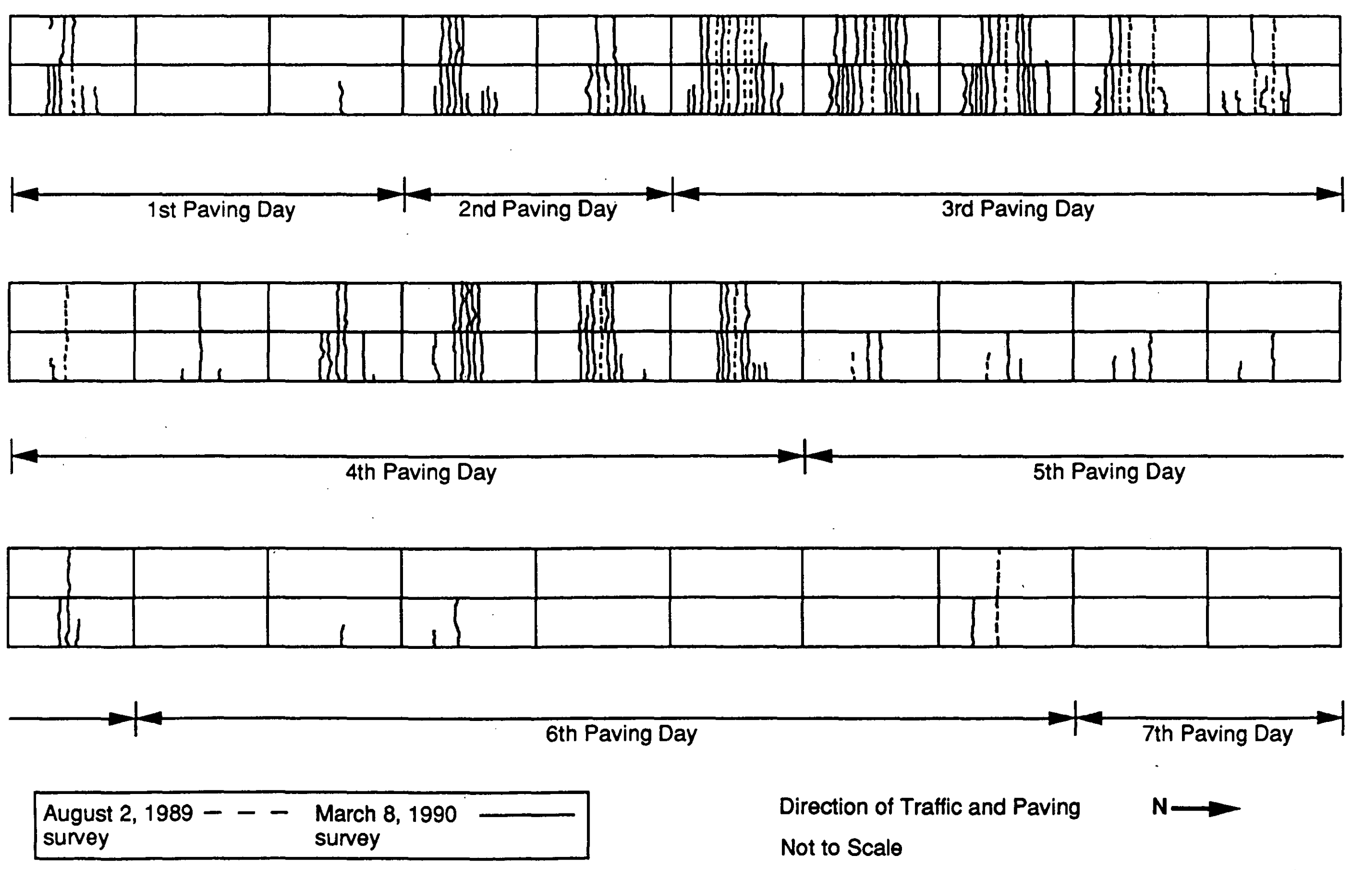

FIGURE 4 Transverse-cracking condition survey.

caused very large shear deformations in the joint sealant at the shoulder joint.

SUMMARY

Long-term monitoring data indicate that at 8 months, concrete creep and shrinkage were about 70 kPa higher than anticipated, assuming no steel relaxation losses. Effective calculated mid-slab prestress using design data for the prestress pavement and increased concrete shrinkage of 200×10^{-6} mm/mm was 393 kPa (permanent). The effective measured mid-slab prestress after 8 months was 517 and 1392 kPa for the two instrumented slabs. A condition survey was performed soon after tied jointed concrete shoulders were placed. Several small transverse hairline cracks near mid-spans were noted in some slabs. A follow-up survey 8 months later indicated a significant increase in transverse cracking. Most of the cracking can be attributed to some effective prestress loss during transfer of prestress and to use of the jointed plain concrete shoulders. As of early 1995, the prestressed pavement section was providing a very good ride, even over sections exhibiting a large amount of cracking. No significant amount of additional cracking had taken place in most of the slabs.

This project has provided verification that prestressed concrete pavements are viable pavement alternatives. The thinner prestressed pavements are promising candidates for unbonded concrete overlays for projects in which grade constraints may not allow thicker conventional jointed concrete pavements.

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