Construction of a Continuously-Reinforced Concrete Pavement in South Dakota

ROBERT A. CRAWFORD and DONALD W. ANDERSON
Assistant Physical Research Engineers, South Dakota Department of Highways, Pierre

•THIS REPORT is primarily concerned with the environment, design, construction, and testing program of two continuously-reinforced concrete test sections constructed in South Dakota in 1963.

The two sections, portions of Interstate Highway Project I90-9(11)395, are located approximately four miles northwest of Sioux Falls and are approximately 3,400 and 4,400 ft long. Figures 1 and 2 show the general location of the project and the location of the test sections within the project.

This project contains two experimental features: (a) the depth of longitudinal steel beneath the surface of the concrete, and (b) the method of controlling end movement. Pavement thickness and quantity of steel were held constant throughout both continuously-reinforced sections.

Section 1 was constructed as part of the eastbound lane and Section 2 the westbound lane. The opposite lanes paralleling the test sections were considered as control sections and were of standard South Dakota design. Both the test control sections were included in the same project and constructed by the same contractor.

Average daily traffic for 1958 was estimated at 4,576 vehicles, and for 1975 it was

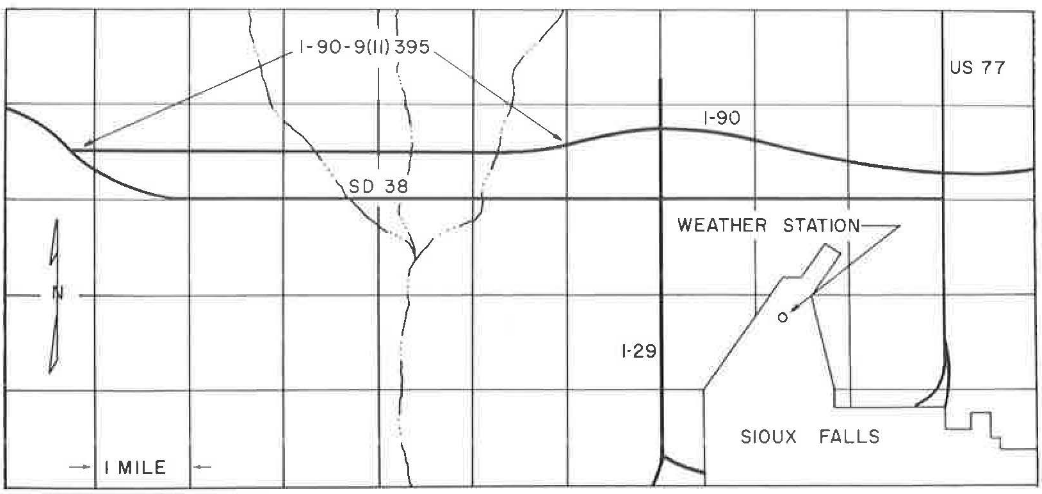

Figure 1. Location of project I-90-9(11)395 containing continuously-reinforced test sections.

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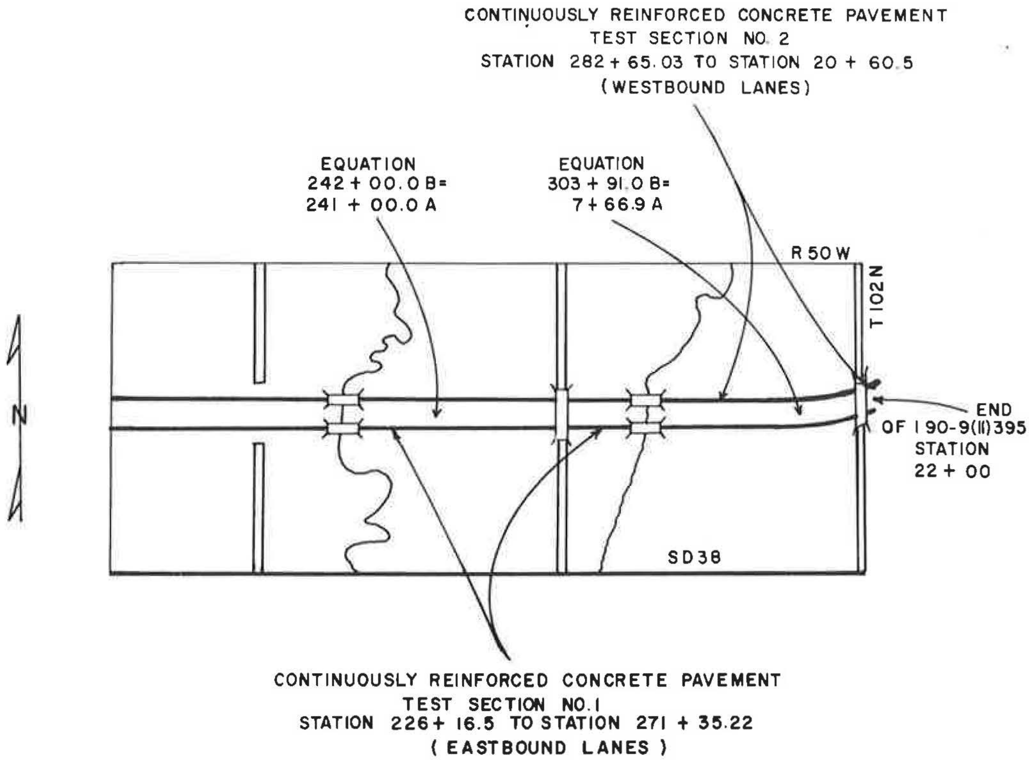


Figure 2. Location of test sections within project.

TABLE 1

TEST SECTION MATERIALS DATA

No. of Samples	AASHO Soil Group and Index	Passing 200 Sieve Range (%)	Passing 200 Sieve Avg. (%)	Liquid Limit Range	Liquid Limit Avg.	Plasticity Index Range	Plasticity Index Avg.	Geological and Pedological Description
25	A-6 (8) to A-7-6(16)	90-100	97	33-47	36	11-26	16	Moody-Nora silty clay loam
5	A-6 (9) to A-6 (11)	98-99	98	34-39	36	13-17	15	Nora-Crofton silt loam
1	A-6 (13)	91	91	40	40	22	22	Rowena silty clay loam
2	A-6 (9)	94-95	94	34	34	14	14	Local alluvial land, fans

(a) Subbase

Gradation Specifications (% passing)							Liquid Limit	Plasticity Index
2-In.	1-In.	¾-In.	¼-In.	No. 10	No. 40	No. 200		
100	80-100	—	40-75	25-55	10-40	0-20	—	Less than 10
—	—	100	50-80	35-65	0-40	0-15	Less than 25	Less than 4

(b) Concrete

Aggregate Type Coarse	Aggregate Type Fine	Max. Size of Coarse Aggregate	Cement Factor	Air Entrained	Slump (in.) Range	Slump (in.) Avg.	Compressive Strength Age (days)	Compressive Strength Range	Compressive Strength Avg.	Flexural Strength Age (days)	Flexural Strength Range	Flexural Strength Avg.
Crushed stone quartzite	Sil. sand	2-in.	1.39 bbl and 1.39 lb pozzolith per cu yd	Yes 3-6%	1¼-3	1¾	28	4,735-6,995	5,771	28	616-783	726

(c) Longitudinal and Transverse Reinforcing Steel

Type	Grade	Bar Size	Tensile Strength Yield Point Range (psi)	Tensile Strength Yield Point Average (psi)	Percent Elongation Range	Percent Elongation Average
Billet	A-432-62T	No. 5	71,780-63,860	67,820	11.72-15.62	13.67

Figure 3. Water standing in poorly drained channel between sections.

estimated to be 10, 799 vehicles per day including 8 percent commercial vehicles. (A 15-min, 16-mm color movie has been made showing all phases of construction. It is available through the Physical Research Division, South Dakota Department of Highways.

ENVIRONMENT

Soils throughout the project area are silt loams and silty clay loams developed from loess. Soil classification shows the majority of them to be A-6 with an occasional A-7-6. Group indexes ranged from 9 to 11. Liquid limits of the soils ranged from 30 to 50. A description of soil samples and their test results are given in Table 1.

Drainage is generally good throughout the test section areas. Natural slopes vary from 1 to 7 percent with the general drainage toward the south. A poorly drained channel crosses the project between the two continuously-reinforced sections and would possibly affect them in periods of above normal precipitation. Figure 3 shows undrained water standing in this channel approximately 24 hr after rain occurred during construction. The east end of Section 2 which had been completed at the time of the rain is visible.

Maximum frost penetration as calculated at the Sioux Falls weather station (3 miles from the project) is 66 in. Average annual precipitation is 25 in. of moisture, including 44 in. of snowfall. Table 2 gives the monthly rainfall, average temperatures, and other climatological data of the period from January 1961 through July 1963. This includes the entire construction period. Table 3 gives the daily weather records for the paving phase.

DESIGN

Subgrade design was according to standard South Dakota practice. Compaction of the soil was to a minimum of 92 percent of Standard Proctor, AASHO Designation T99-57, Method A, with the upper 12 in. of subgrade requiring a minimum density of 96 percent. Moisture content was held to within 2 percentage points above and 6 percentage points below optimum moisture.

Two types of subbase were used for a combined thickness of 9 in. throughout the continuously-reinforced sections. The lower 6 in. was 2-in. maximum size and the upper 3 in., "subbase cushion," was $^3/_4$-in. maximum size. Subbase specifications are given in Table 1. The material was produced from a local gravel pit and was manufactured largely by screening with a small amount of crushing. Tests on the material as it was placed on the project showed compliance with specifications. Compaction of the subbase was to a minimum density of 95 percent Standard Proctor, AASHO Designation T99-57 Method D.

For this project, a pavement thickness of 8 in. was chosen for the continuously-reinforced sections using concrete with a compressive strength of 5, 000 psi and a flexural strength of 600 psi. The conventional pavement in the control sections had a thickness of 9 in. and used concrete of 3, 000-psi compressive strength. Coarse

TABLE 2

SIOUX FALLS AREA CLIMATOLOGICAL DATA

Time Period	Max. Temp.	Avg. Daily Max. Temp.	Min. Temp.	Avg. Daily Min. Temp.	Avg. Monthly Temp.	Monthly Departure from Normal	Precipitation
Jan. 1961	45	25.7	-23	2.7	14.2	0	0.25
Feb. 1961	51	34.2	-13	12.9	23.6	+4.1	0.92
Mar. 1961	67	43.9	18	27.3	35.6	+3.6	1.14
Apr. 1961	81	53.3	17	30.4	41.9	-4.5	1.04
May 1961	86	65.7	25	44.2	55.0	-3.1	4.67
June 1961	98	80.9	43	56.7	68.8	+0.8	3.86
July 1961	95	84.7	52	59.6	72.2	-2.6	2.16
Aug. 1961	98	86.6	51	61.4	74.0	+1.6	1.79
Sept. 1961	92	70.5	28	48.0	59.3	-3.1	2.36
Oct. 1961	84	64.5	22	38.1	51.3	+1.3	2.66
Nov. 1961	62	42.8	13	23.9	33.4	+1.2	1.40
Dec. 1961	56	25.9	-16	6.1	16.0	-3.4	0.80
Jan. 1962	47	23.2	-22	2.3	12.8	-2.4	0.29
Feb. 1962	57	25.4	-31	9.0	17.2	-1.9	4.05
Mar. 1962	55	32.1	-16	17.1	24.6	-5.5	1.72
Apr. 1962	94	57.9	18	33.5	45.7	-0.2	1.70
May 1962	88	73.6	36	52.4	63.0	+4.7	6.07
June 1962	91	77.5	45	57.7	67.6	-0.5	3.98
July 1962	94	80.5	47	60.8	70.7	-3.6	5.50
Aug. 1962	94	83.8	48	59.3	71.3	-0.5	2.77
Sept. 1962	85	71.3	32	47.4	59.4	-2.4	3.58
Oct. 1962	82	64.6	19	40.7	52.7	+2.4	0.46
Nov. 1962	74	49.0	15	28.2	38.6	+6.0	0.16
Dec. 1962	61	33.0	-14	12.3	22.7	+1.6	0.19
Jan. 1963	54	16.6	-22	-3.3	6.7	-8.5	0.90
Feb. 1963	59	30.9	-13	8.7	19.8	+0.7	0.53
Mar. 1963	80	49.8	16	28.1	39.0	+8.9	1.16
Apr. 1963	86	61.5	20	37.3	49.4	+3.5	1.25
May 1963	89	70.6	27	48.2	59.4	+1.1	2.00
June 1963	99	85.0	50	62.0	73.5	+5.4	2.51
July 1963	98	87.1	58	64.5	75.8	+1.5	6.45

TABLE 3

CONSTRUCTION PERIOD WEATHER DATA

Date 1963	Temperature		Precipitation (in.)	Wind Average (mph)	Sky Cover (tenths)	Construction Operation
	Max.	Min.				
5-25	70	54	Trace	14.5	9	Poured Section 2
5-26	72	57	Trace	13.5	10	Curing
5-27	66	54	Trace	12.0	10	Curing
5-28	74	50	Trace	12.0	5	Curing
5-29	81	44	0	9.2	1	Poured east end Section 1
5-30	87	51	0.20	11.9	9	Curing (holiday)
5-31	71	62	0.12	19.7	10	Completed Section 1
6-1	74	60	0.28	13.5	8	Curing
6-2	82	58	0.14	9.1	8	Curing
6-3	85	63	0	14.9	5	Curing

aggregate was 100 percent crushed quartzite, and the fine aggregate consisted of silicious sand having an average fineness modulus of 2.85. Specifications required 3 to 6 percent air entrainment and a slump of between 2 and 3 in. Type 1 cement was used for all concrete. The contractor on this project elected to use pozzolith, and job mixes using this admixture were designed. General information concerning the continuously-reinforced concrete is given in Table 1. The job mixes for the continuously-reinforced and conventional pavements within this project are given in Table 4.

Because of severe climatic conditions, it was decided to use over 0.6 percent longitudinal steel. This requirement was met with No. 5 deformed bars spaced on 6-in. centers. This provided longitudinal steel at the rate of 0.646 percent of the cross-sectional area of the slab. A further safety factor was the use of A-432 billet steel with a minimum yield point of 60,000 psi (Table 1). Specifications required the steel in 40-ft lengths as it was felt that this was the maximum length which could be easily handled. Transverse steel was also No. 5 deformed bars of the same billet steel spaced on 44-in. centers, 23 ft 10 in. long for a 24-ft pavement width. All longitudinal and transverse steel intersections were tied. Support chair placement under the intersection of every sixth longitudinal bar with every transverse bar provided one support chair ever 1.222 sq yd.

TABLE 4

CONCRETE MIXTURE DATA

Material	Continuously-Reinforced	Conventional
Cement	523 lb	478 lb
Water	232 lb	225 lb
Fine aggregate	1,166 lb	1,216 lb
Coarse aggregate (3/4-No. 4)	1,209 lb	1,211 lb
Coarse aggregate (1½-3/4)	806 lb	807 lb
Air-entraining agent	1.85 oz	1.27 oz
Pozzolith	1.39 lb	1.27 lb

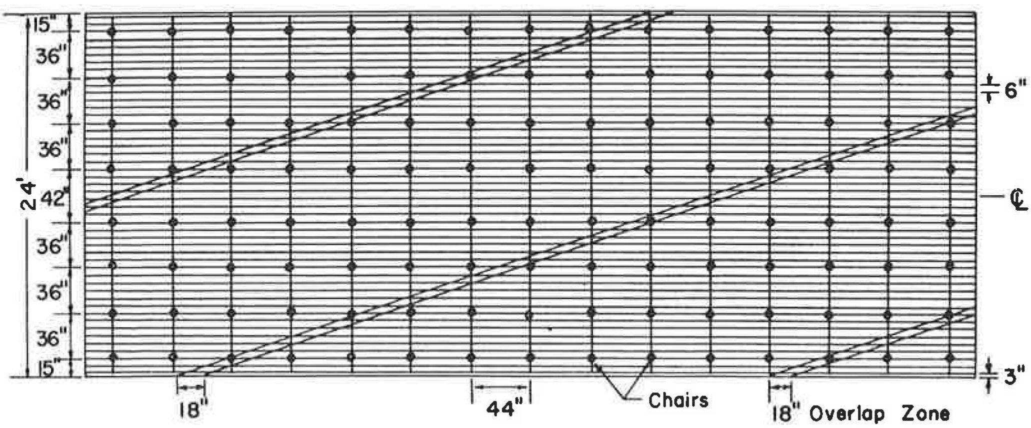
Figure 4. Support chairs for reinforcing steel.

Specifications required that the chairs (Fig. 4) be approved by the State engineers because it was important that the steel be maintained in its proper position during concrete placement.

An 18-in. overlap was specified for the longitudinal steel (approximately 29 diameters). To strengthen the pavement further, a staggered lap lime was chosen—the end of one 18-in. lap being even with the beginning of the adjacent lap. This resulted in the lap line being skewed approximately 18^0 from the centerline. All overlaps were tied twice to insure against slippage while placing concrete. Figure 5 shows the layout of the steel and the overlaps.

As a research feature, the depth to the center of the longitudinal steel was $2\frac{1}{2}$ in. in Section 1 and $3\frac{11}{16}$ in. in Section 2. The purpose of this variation was to determine if the steel depth has any effect upon crack pattern, depth, or width. A construction tolerance of $\pm\ {}^3/_8$ in. was allowed in both horizontal and vertical placement of the longitudinal reinforcing steel.

A frequent failure location in continuously-reinforced concrete is at the construction joints. To prevent failure and to furnish additional load transfer, additional steel was specified for construction joints. Longitudinal bars that did not extend 4 ft into the



LONGITUDINAL STEEL – No. 5 x 40' Billet Steel A432 – 48 Deformed Bars for 24' Width Provides 0.646 % of Cross Sectional Area

TRANSVERSE STEEL – No. 5 x 23' 10" Billet Steel A432 Deformed Bars Spaced on 44" Centers

CONCRETE – 8" Thick, 1 1/2" Max. Size Aggregate. 5000 P.S.I. Compressive Strength, 600 P.S.I. Flexural Strength at 28 Days

DEPTH OF LONGITUDINAL STEEL – Section 1 – 2 1/2" and Section 2 – 3 11/16" From Surface of Slab to Center of Longitudinal Steel

OVERLAP OF LONGITUDINAL STEEL – 18" Overlap Staggered as Shown (18" = 28.8 Diameters)

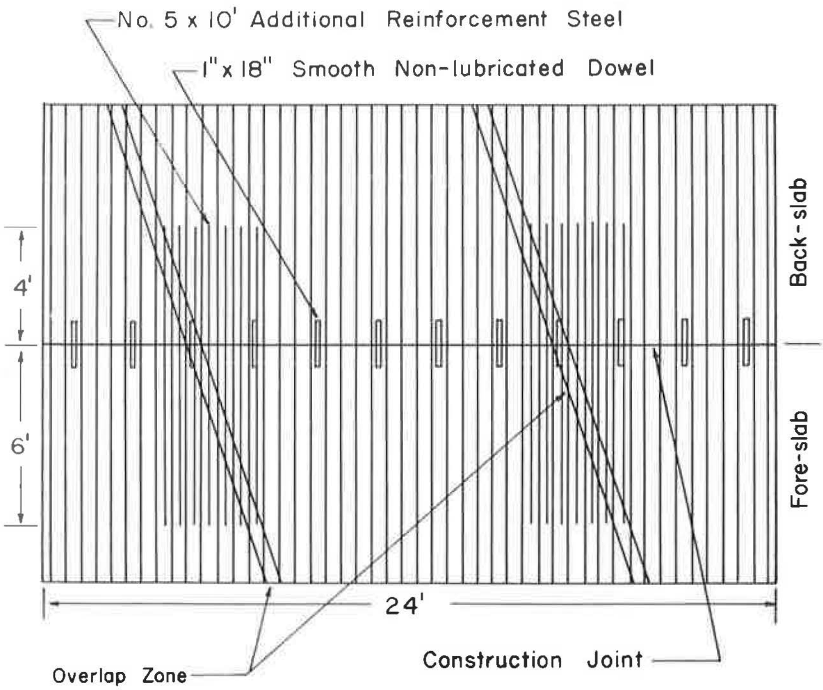
SPECIAL SUPPORTING CHAIRS – One Per 1.222 Sq. Yd.

Not To Scale

Figure 5. Plan showing steel and overlap layout.

back-slab or 6 ft into the fore-slab were spliced with additional 10 ft long No. 5 bars placed to extend 6 ft into the fore-slab. In addition, 18 in. long 1-in. dowel bars were placed on 24-in. centers, starting 12 in. from the edge of the slab. Figure 6 shows the steel placement at a construction joint.

Two different types of terminal joints were used (transverse lug end anchors and wide-flange joints). The wide-flange joints were used in Section 1 and consisted of a 10-in. wide-flange beam weighing 100 lb/ft and shaped to conform to the crown of the pavement. The beam was placed in a 8-in. thick reinforced-concrete sleeper slab. Figure 7 shows the I-beam and reinforcing steel in place prior to pouring the sleeper slab, and Figure 8 shows continuously-reinforced concrete in place against the beam. Steel plates were welded to each end of the beam to prevent the intrusion of subbase material between the beam and the pavement during cold weather pavement contraction. Figure 9 shows the design details of the sleeper slab and I-beam. The portion of the sleeper slab upon which the continuously-reinforced pavement rested was finished as smoothly as possible to prevent interlock of the pavement and the sleeper slab. Friction



All No. 5 Deformed Longitudinal Bars Which do not normally extend at least 6 feet into the Fore-slab beyond a Construction Joint, or 4 feet into the Back-slab, are spliced with No. 5 deformed bars, 10 feet long, extending 6 feet into the fore-slab.

In addition, the Construction Joint is reinforced with 1" x 18" smooth dowels, non-lubricated, spaced 24" center to center and 12" from edge of pavement, and at one-half the depth of pavement.

Splice bars and dowels are of billet steel.

Figure 6. Construction joint design.

was further reduced by a coating of asphalt placed upon that side of the I-beam and slab prior to pouring the pavement. A rougher, nonlubricated surface was constructed on the side of the sleeper slab that held the conventional pavement.

Three end-anchor lugs were placed at each end of Section 2. The reinforced end anchors, 5 ft deep and 2 ft thick, were cast separately to the full width of the pavement. A keyway was formed into the top of each lug to aid in transmitting pavement stresses

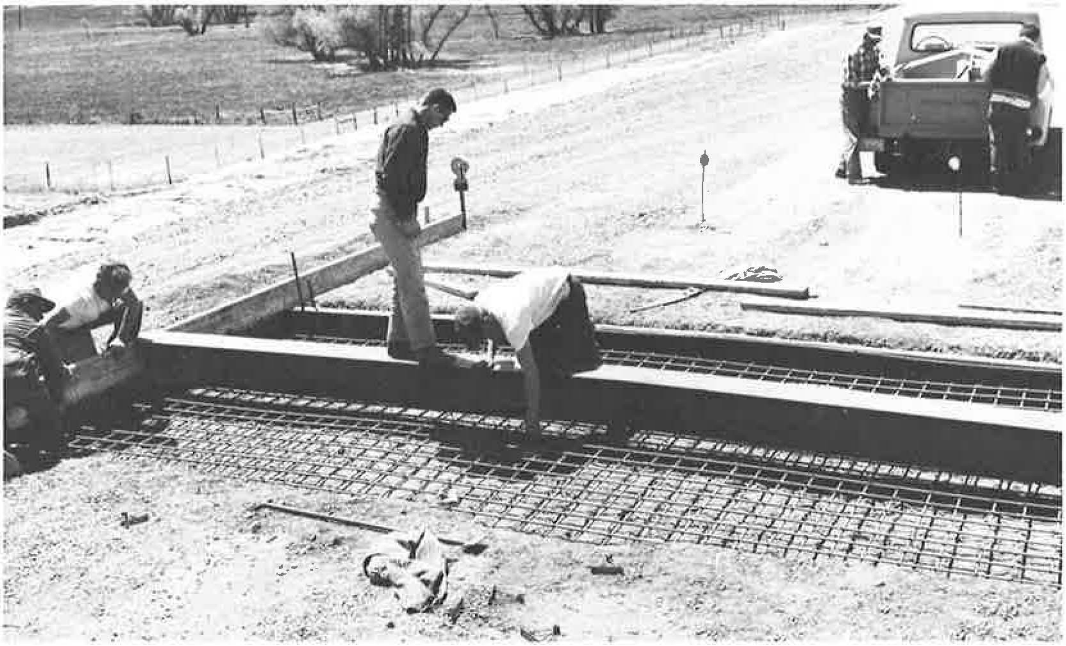


Figure 7. Wide-flange end joint prior to pouring sleeper slab.

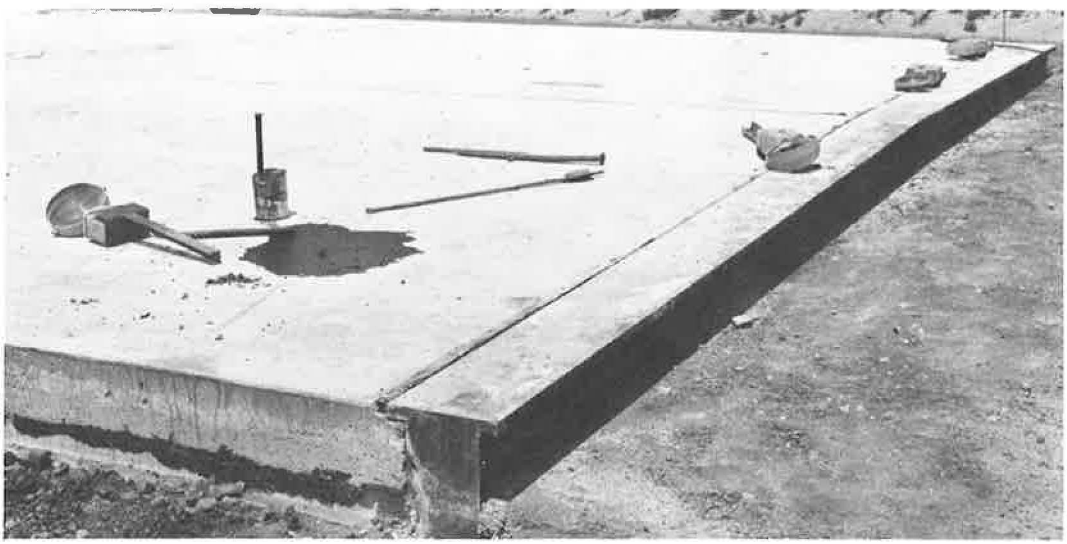


Figure 8. Continuously-reinforced concrete in place against wide-flange beam.

to the anchor lug. A completed end-anchor lug is shown in Figure 10. The pavement thickness was increased to 10 in. in the region of the end anchor lugs. Figure 11 shows design details of the end anchors.

Three expansion joints with 46.5-ft spacing were placed in the conventional pavement at the ends of each section. These joints consisted of 1 in. of mastic with lubricated dowel bars $1\frac{1}{4}$ in. by 18 in. long spaced on 12-in. centers. The top of the joint was sealed with rubber asphalt filler.

As previously mentioned, the control sections were designed and built according to standard South Dakota pavement designs. Subgrade and subbase requirements were the same as the continuously-reinforced sections with the exception of subbase thickness. Standard design procedures specified different subbase thicknesses in the control sections, while a uniform 9-in. thickness was used in the test sections to limit the variables.

Figure 12 shows the total subbase thicknesses of both pavement designs. Conventional designs specify a 9-in. pavement thickness with welded-wire fabric reinforcement placed $2\frac{1}{2}$ in. below the surface. The reinforcement must weigh not less than 60 lb per 100 sq ft and consists of No. 1 longitudinal wires on 6-in. centers with No. 4 transverse wires on 12-in. centers. The ends of the mesh are lapped 12 in. Concrete having a compressive strength of 3,000 psi is used. Thirty-inch long No. 5 bars are placed every 48 in. across the pavement centerline as tie bars. One and one-fourth inch diameter are placed mid-depth in the pavement on 12 in. centers at contraction joints. These transverse joints are spaced at 46 ft 6 in. Expansion joints are normally specified only at structures.

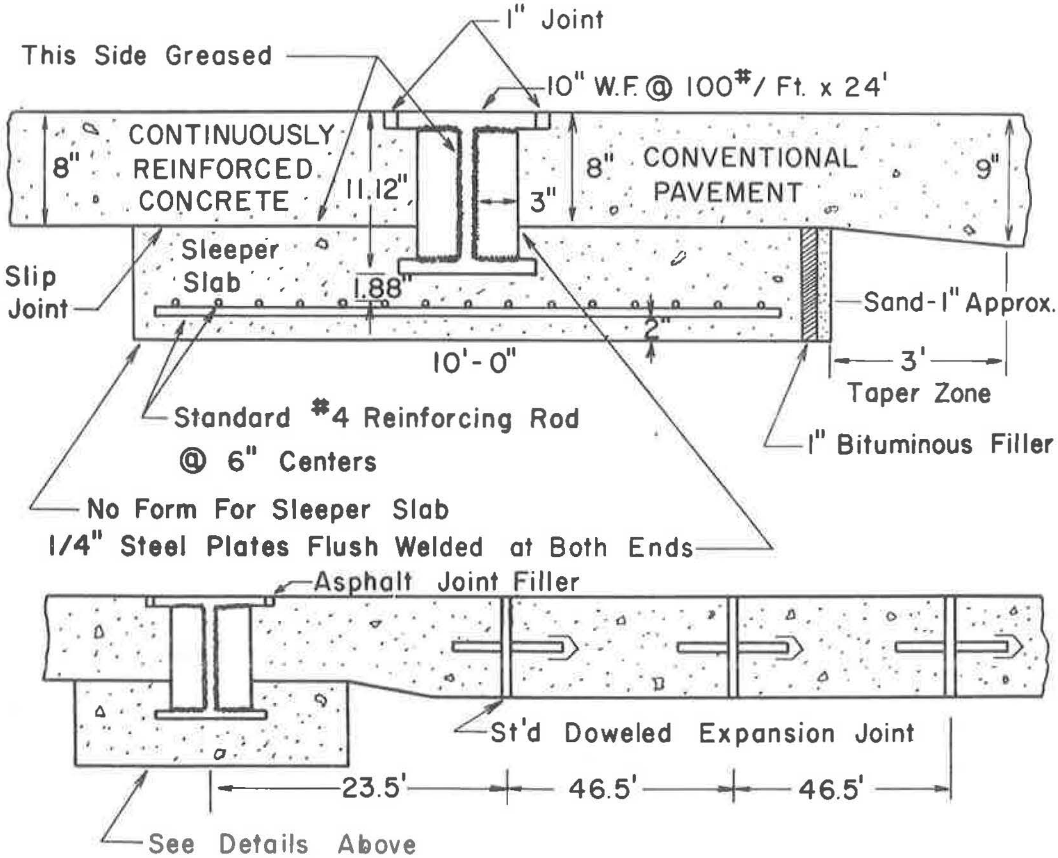


Figure 9. Wide-flange joint design details.

Figure 10. Completed end-anchor lug.

The longitudinal joint was sawed to one-fourth the depth of the pavement for both the conventional and continously-reinforced sections. Transverse joints on the conventional pavement were formed with steel contraction joint inserts that were later crimped for placement of the sealing compound.

Conventional shoulder design was used throughout a 4-ft inner and 10-ft outer shoulder, and both were covered with a 2-in. thickness of bituminous material.

CONSTRUCTION

Grading operations were carried out in 1961 and 1962. No unexpected difficulties were encountered, and this work was completed on schedule. The subbase was placed during the fall of 1962. Depth checks made subsequent to this showed plan thicknesses to be present.

The end-anchor lugs and I-beam sleeper slabs were constructed approximately a week prior to pouring the pavement. The lugs were simple to construct because no forms were required except to shape the top of the lug.

In order to maintain the wide-flange beam at the proper elevation in the sleeper slab, the contractor tack-welded the ends of the beam to paving forms that had previously been placed to grade (Fig. 7). The reinforcing steel was then placed under the beam, tied, set on chairs, and the concrete poured.

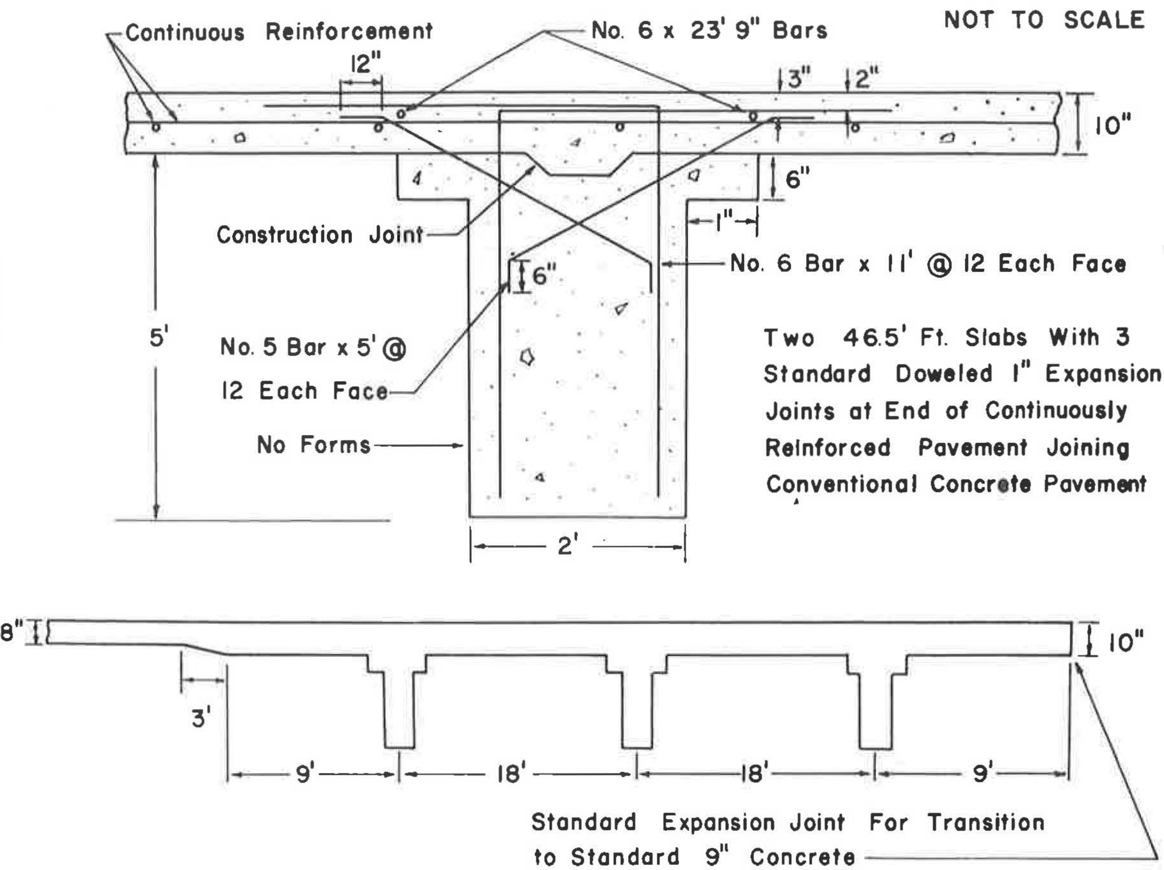


Figure 11. Design details of transverse lug end anchorage.

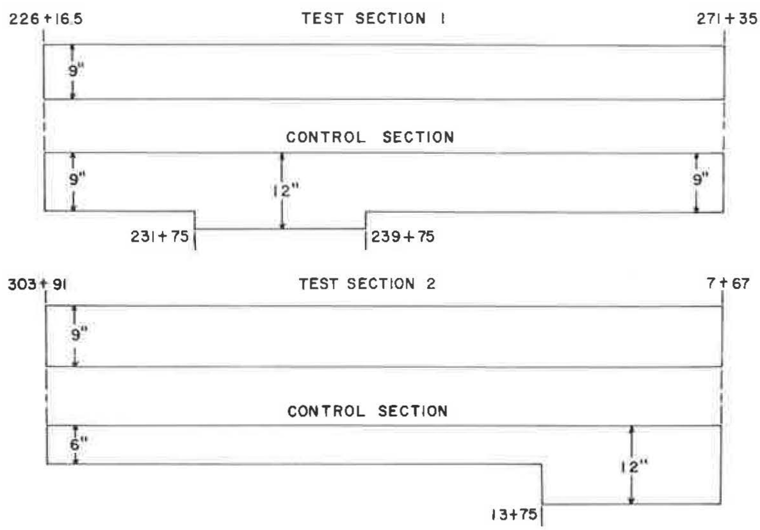


Figure 12. Total gravel subbase thickness for both pavement designs.

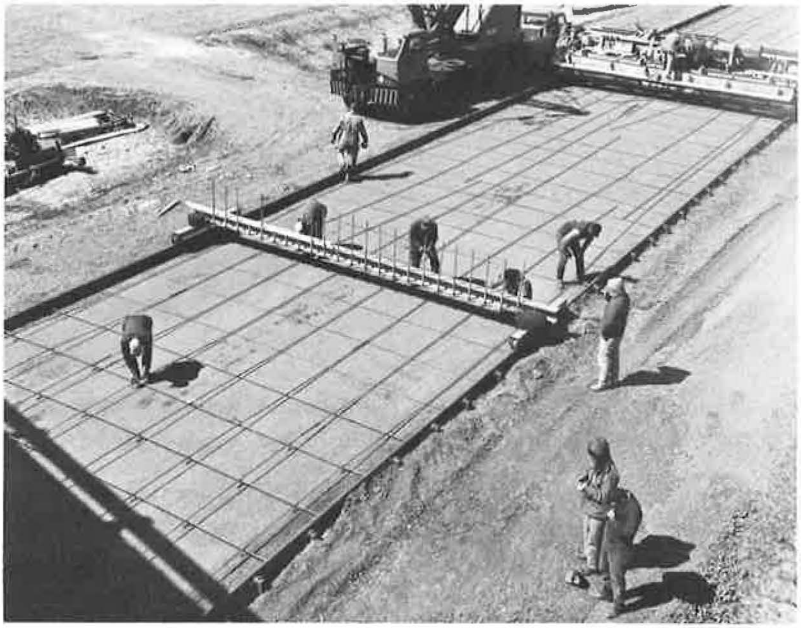


Figure 13. Reinforcing steel operational sequence.

Fine-grading was completed and the forms were set in place before any steel was placed. The forms were then marked and the transverse steel was laid in position on the grade. Every sixth longitudinal bar was laid on top, and both the overlaps of this longitudinal steel and its intersections with the transverse steel were tied. Chairs were then placed under the transverse steel to lift it to its proper elevation. Figure 13 shows this operational sequence. The first eight longitudinal bars were spaced using steel rods welded on an old scratcher. The remaining bars were then carried into place and tied, spacing them from the original eight bars. As previously mentioned, all transverse and longitudinal steel intersections were tied, and all overlaps were tied twice. Figure 14 shows the steel in place before pouring concrete in Section 1.

Section 2 was paved on May 25, 1963. The paving train consisted of two 34-E dual-drum pavers, one spreader, one screed, one bull float, two finish men, a belting machine, and a burlap drag. Ten spud-type vibrators attached to the front of the screed were used to consolidate the concrete. Care was taken to prevent contact of the vibrators with the reinforcing steel because this would dampen their effect.

Section 1 was paved on May 29 and 31. The paving train was identical with the exception of an additional screed and the use of a pan vibrator instead of the spud-type vibrators. This was found to be necessary in this section because of the shallower steel that would not permit full immersion of the spud vibrators within the concrete.

Curing was accomplished with reflective plastic sheeting placed upon the slab for a minimum of 72 hr. The weather during the paving and curing period was variable (Table 3).

No movement of the steel was noticeable while placing the concrete. One 100-ft long section at station 17+00 to 18+00 where only every other transverse and longitudinal steel intersection was tied was observed to move excessively. This tie arrangement was used to see if it was possible to reduce the steel tying labor. When movement was observed, the practice was discarded. No difficulty was encountered in getting the concrete to flow around the reinforcing steel.

Figure 14. Reinforcing steel in place.

The stationing was marked in the pavement every 100 ft to facilitate observation and testing after construction.

Hourly temperature measurements were taken of the concrete as it left the paver. A maximum temperature of 72 F and a minimum of 60 F were recorded with an average concrete temperature of 66 F. Although the weather resulted in above normal temperatures during the curing period, the plastic during membrane reflected sufficient heat to keep the pavement quite cool. Upon removal of the plastic after 72 hr, there were no visible cracks other than a few shrinkage cracks. Transverse pavement cracking first began to appear the day after the curing plastic was removed.

TESTING PROGRAM

In addition to the previously mentioned construction testing, a testing program of indefinite duration is to continue. This program includes crack-width measurements, end movement surveys, and present serviceability index studies.

As rapidly as cracks developed, holes were drilled on each side of the crack and brass plugs cemented into the holes. These plugs were then drilled so that measurements of crack width and movements could be made with a 10-in. Whittemore strain gage. A minimum of 12 selected cracks in each end and central 500-ft section of both

TABLE 5

MAXIMUM CRACK WIDTH MEASUREMENTS

West End (in.)	Center (in.)		East End (in.)
Section 1 (avg. 0.008 in.)			
0.009	0.005	0.006	0.010
0.007	0.007	0.007	0.010
0.004	0.013	0.006	0.010
0.012	0.006	0.009	0.005
0.010	0.008	0.003	0.011
0.012	0.005	0.006	0.012
0.012	0.007		0.008
0.009 average	0.007 average		0.009 average
Section 2 (avg. 0.011 in.)			
0.008	0.010	0.006	0.010
0.015	0.012	0.014	0.011
0.007	0.015	0.010	0.021
0.012	0.012	0.009	0.013
0.013	0.013	0.010	0.009
0.011	0.007	0.010	0.013
0.010	0.009		0.006
0.011 average	0.011 average		0.012 average

continuously-reinforced sections will be measured. These measurements will be made during August and January of each year when the air temperatures are over 100 F and less than -20 F, respectively. Crack location surveys will be conducted in the same 500-ft sections during September, December, March, and June during the first year after construction and each fall thereafter.

Permanent reference points free from frost action have been placed at the ends of each continuously-reinforced section, and measurements will be made of pavement end movements during the hottest and coldest periods of each year to determine the range of end movement. The measurements will be made from the reference points with a transit, and the distance from the line of sight to a permanently established point in the pavement, approximately 6 in., will be measured with a ruler graduated to 0.02 in. In addition to continuously-reinforced sections' end movement, movement of the sleeper slab I-beam and the conventional pavement at the ends of Section 1 will also be measured.

Pavement temperatures will be taken at the time of the crack measurements. These temperatures will be obtained from mercury-filled temperature wells that were installed during pavement construction.

The present serviceability indexes of the test and control sections will be determined annually with a BPR-type roughometer. This roughometer has been correlated with CHLOE profilometers and a PSI formula developed. Traffic counts and axle-load weights will be obtained once each year.

In August 1963, four cores were drilled over cracks in the test sections. It was found that three of the four cracks followed transverse reinforcing bars. Longitudinal steel was found to be within ¼ in. of the plan depth within the slab. There was no apparent difference in the quality of vibration produced by the spud and pan vibrators. Some porosity was found in cores which had been vibrated by either method.

Although the crack patterns are not yet fully developed, a slight difference between the two sections is apparent. In Section 1, with the shallower steel, the average crack spacing exclusive of the section ends is approximately 2.3 ft. The majority of the cracks in this section are on a 1- to 2-ft spacing with occasional unbroken pavement 10 to 15 ft in length. The influence of the unrestrained pavement ends is shown by a diminishing crack pattern starting approximately 120 ft from the east end and 500 ft from the west end of Section 1. There are no cracks in the final 100 ft of either end of this section. Maximum measured movement of the pavement ends at the I-beams has

been 0.074 in. at the east end and 0.095 in. at the west end with a pavement temperature of 2 F.

Section 2 with the steel placed nearly at mid-depth has an average crack spacing of 3 ft. The crack frequency is somewhat less within 100 ft of the end-anchor lugs because of slight movement of the lugs. Measurements have shown this movement to be 0.068 and 0.046 in. on the east and west ends of this test section, respectively. However, cracks are present up to and between the anchor lugs.

Crack widths are small in both sections. The crack widths as measured by a 10-in. Whittemore straingage are given in Table 5. The values listed are for the measured widths of 27 selected cracks at the 500-ft end and central portion of each test section. These measurements represent the maximum reading obtained for each crack since test section construction.