HIGHWAY RESEARCH BOARD

Bulletin 274

Concrete Pavement Design And Performance Studies: 1960

National Academy of Sciences—
National Research Council

publication 812

HIGHWAY RESEARCH BOARD

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1960

NRC HIGHWAY RESEARCH BOARD

Bulletin 274

Concrete Pavement Design And Performance Studies: 1960

Presented at the

39th ANNUAL MEETING

January 11-15, 1960

1960
Washington, D.C.

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Contents

EXPERIENCE IN TEXAS WITH CONTINUOUSLY-REINFORCED CONCRETE PAVEMENT

M.D. Shelby and B.F. McCullough 1
Appendix 22

AN EXPERIMENTAL CONTINUOUSLY-REINFORCED CONCRETE PAVEMENT IN MICHIGAN

Gene R. Cudney 30
Appendix A 50
Appendix B 51
Appendix C 53

THE PROBLEM OF CORROSION OF LOAD TRANSFER DOWELS

Robert G. Mitchell 57

INVESTIGATIONAL CONCRETE PAVEMENT IN MINNESOTA: 18-YEAR REPORT

P.G. Velz and E.C. Carsberg 70

EUROPEAN DEVELOPMENTS IN PRESTRESSED CONCRETE PAVEMENTS FOR ROADS AND AIRPORTS

Armand Mayer 92
Discussion: Phillip L. Melville and Paul F. Carlton 109
Closure: A. Mayer 110

THE CASE FOR SKEWED JOINTS

R.H. Cooley 113

AN EXPERIMENT IN PAVEMENT SLAB DESIGN

W.E. Chastain, Sr. and John E. Burke 119

COLORADO CONCRETE PAVEMENT AND SUBBASE EXPERIMENTAL PROJECT

Charles Lowrie and W.J. Nowlen 150

Experience in Texas with Continuously-Reinforced Concrete Pavement

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This paper summarizes the results of nine years of experience with continuously-reinforced concrete pavement in Texas. The reasons for trying this type of pavement are presented. Data and conclusions based on physical measurements and observations of new and old projects are reviewed, and the possible reasons for variations of the crack patterns are discussed.

With the above data and observations, and reported experience in other states as a background, the current design policies and practices followed by the Texas Highway Department in the design and construction of continuously-reinforced concrete pavements are enumerated.

In addition a short summary of projects constructed and proposed for construction is reported. The tables in the summary cover the characteristics peculiar to each project such as the use of air, use of concrete with a low modulus of elasticity, various pavement thickness, and variations of bond area for a given percent of steel.

● SINCE THE inception of the Texas Highway Department, numerous jointed concrete pavements have been constructed in various sections of the state. Thickness of pavement from 6 to 12 in. along with various spacings of expansion and contraction joints have been used during the period. For a considerable period of time designers favored the thickened edge section (9-6-9). At the present time, a uniform cross-section thickness is generally used.

In a jointed pavement, the joint itself presents the most troublesome feature from both the standpoint of maintenance and riding quality. Proper joint spacing using corrugated metal contraction joints has resulted in improved riding quality, but proper construction methods are a must in the installation of this joint type. Because the nature of high traffic volume facilities, such as freeways, precludes frequent interruption in traffic service by routine maintenance operations, a pavement type having only a minimum number of joints was sought. Experience with continuously-reinforced concrete pavement constructed in other states indicated that this type of pavement exceeded all expectations both service wise and maintenance wise. This background of experience rather naturally resulted in the experimental use of a continuously-reinforced concrete pavement in the City of Fort Worth. The excellent performance on this first project has resulted in the construction of a number of other projects with certain indicated changes in the design.

This paper discusses the data obtained from observations on two continuously-reinforced pavements constructed in Texas, the present design procedures of the Texas Highway Department, and concludes with a summary of outstanding features on several projects over the state. Discussions of data based on observations and measurements are presented for a Falls-McLennan County Project and for several Tarrant County

Projects. The recent Falls-McLennan Project is presented before the more widely known Tarrant County Projects because more is known about this project from its inception to its completion. The design procedure for continuously-reinforced pavement is presented in three basic steps. The summary of recent projects using this type of pavement discusses experience with low modulus concrete and variations in bond area for an equivalent percentage of steel. Along with the summary is a tabular list of the continuously-reinforced concrete projects contracted for construction in Texas.

FALLS-McLENNAN COUNTY PROJECT

General

This project was the first continuously-reinforced concrete pavement constructed under the recommended Texas Highway Department design policy of providing for 0.5 percent longitudinal steel. The project consists of 6 mi of Interstate Highway 35 in a rural area about 15 mi south of Waco, Texas. The highway is the divided type consisting of two lanes in each direction. A typical section for this project is shown in Figure 1. Slab lengths between expansion joints on this project ranged from 1,100 ft to 21,700 ft (Table 1). Expansion joints were used at structure ends only. Continuously-reinforced concrete was used on the main lanes, the speed change lanes, and the ramps.

All but 5,500 ft of the northbound lane consists of an 8-in. uniform pavement placed on a 10-in. granular base course. The exception was where an existing 9-6-9 concrete pavement 5,500 ft long with 3.5 in. of hot mix asphaltic concrete level-up was overlaid with a minimum 7 in. of continuously-reinforced pavement. Due to the special considerations involved, data pertaining to this section of overlay will not be presented in this report.

The north ends of the pavements join a standard 10-in. corrugated metal joint concrete pavement with joints on 15-ft centers. The south ends of the pavements join a flexible-type pavement. The shoulders both inside and outside were sealed with 1½ in. of hot mix asphaltic concrete. The inside shoulder is 6 ft wide and the outside is 10 ft wide.

The density of the subgrade and of the base was controlled by the Compaction Ratio Method as outlined in THD 110 Procedure of the Texas Highway Department Soil Testing Procedure (1). The natural material consisted of a black waxy soil (P.I. of about 40) underlain by the Austin Chalk. Crushed limestone and gravel were used as the subbase material (Project No. 7, Table 3).

Design and Construction Procedures

Concrete. —The concrete was mixed and placed in accordance with the standard Texas Highway Department specifications, Item 320, for concrete pavement (2). Table 2 gives a few of the pertinent requirements for the concrete compared to the values

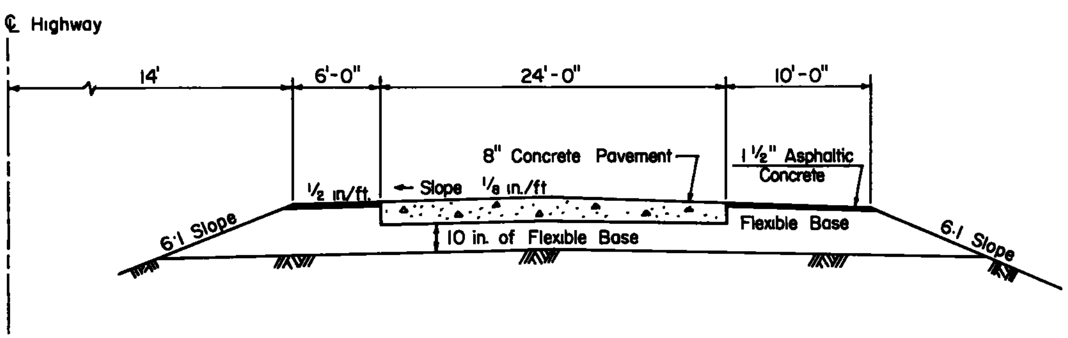

Figure 1. Typical half section for Falls-McLennan County Project.

TABLE 1

STATIONING OF THE SLAB TERMINALS

Project	Lane	Beginning of Slab	End of Slab	Length (ft)
Falls and McLennan Co.	Northbound	0+00	216+99.4	21,699.4
	Southbound	0+00	217+15.0	21,715.0
	Northbound	222+38.4	280+65.0	5,826.4
	Southbound	222+99.6	280+83.2	5,783.6
	Northbound	282+60.0	294+10.0	1,150.0
	Southbound	282+78.2	294+10.0	1,131.8
	Northbound	296+20.0	326+06.0	2,986.0
	Southbound	296+20.0	326+06.0	2,986.0
Tarrant Co.	Northbound and southbound	252+78.6	298+07.8	4,529.2
	Northbound and southbound	300+51.4	323+26.3	2,274.9
	Eastbound and westbound	2+41.2	79+22.6	7,681.4
	Eastbound and westbound	83+69.6	104+54.2	2,084.6
	Eastbound and westbound	104+54.2	133+67.9	2,913.7
	Eastbound and westbound	141+16.7	162+22.4	2,105.7
	Eastbound and westbound	164+35.6	170+06.3	570.7
	Eastbound and westbound	173+17.3	195+42.2	2,224.9

TABLE 2

COMPARISON OF SPECIFICATION REQUIREMENTS WITH VALUES OBTAINED IN THE FIELD

Items	Unit	Values Required By Specifications	Values Obtained in the Field: Falls-McLennan Co.	Tarrant Co.
Water-cement ratio	Gal. per sack	6.5 or less	6.2-6.5	6.3-7.2
Cement factor	Sacks per cu yd	5 or greater	5	4.7-5.3
Coarse aggregate factor	-	0.85 or less	0.78	0.80-0.85
Seven-day flexural strength	Psi	650 or greater	650-920	575-750
Slump	In.	1-3	2-2½	1½-2½

obtained in the field. Although the specifications required the use of either Polyethylene Film Blankets, cotton mats, or waterproofed paper for curing, a field change permitted the substitution of the "Hunt's Process." Spud vibrators on the front of the finishing machine were used to consolidate the concrete. No attempt was made to determine the tensile strength of the concrete, but the normal procedure of requiring two flexural tests per 500 sq yd of pavement was followed using mid-point loading.

The pavement placement for the main lanes was 24 ft in width. Speed change lanes were poured separately after a time interval of 2 to 30 days. A longitudinal plane of weakness was provided between the main lanes. All longitudinal and transverse construction joints were of the tongue and groove type. Figure 2 shows the grooved section of a typical construction joint before the pour on the adjacent slab was initiated. Expansion joints of a special design were used at the end of all bridges (Fig. 3). This joint was an experimental design, and the construction of it presented several problems which probably preclude its future use.

Steel.—The plans called for the longitudinal steel to be ¾-in. round, hard grade steel bars placed at 11.25-in. centers and the transverse steel to be ½-in. round bars placed at 24-in. centers. The steel reinforcement was assembled in place in the field (Fig. 4). Specifications called for the preplacement of the steel before pouring of the concrete with the longitudinal steel placed at mid-depth. The longitudinal bars were usually 40 ft long with a tied lap splice of 20 diameters. Reinforcing bars were held at the desired depth by means of simple L-shaped, ¼-in. bar chairs. These chairs were welded to the transverse bars on 30-in. centers, and proved to be adequate to support the steel on the granular subbase. The steel across the longitudinal joint consisted of ½-in. round bars of structural grade.

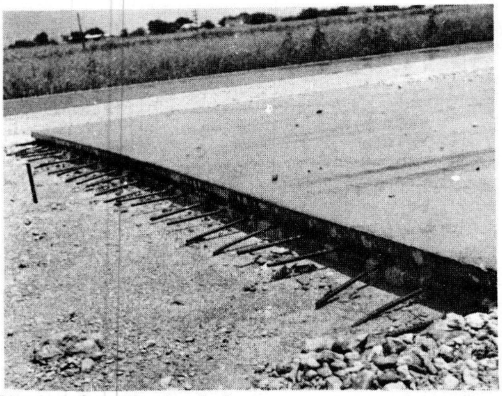

Figure 2. Typical transverse construction joint for Falls-McLennan County Project.

Performance of the Pavement

The paving operations commenced in January 1959 and continued to June 1959. Traffic was turned on most of the pavement during the latter part of June, and since that time it has been subjected to an average daily traffic of 5,700 vehicles per day.

The pavement has been under relatively close observation since the initial pouring operations in order to ascertain the cracking pattern as it occurs. Crack surveys have been taken at frequent intervals since the initial construction operations.

Crack Spacing

Surveys of the crack spacing indicated that the pavement had several crack patterns

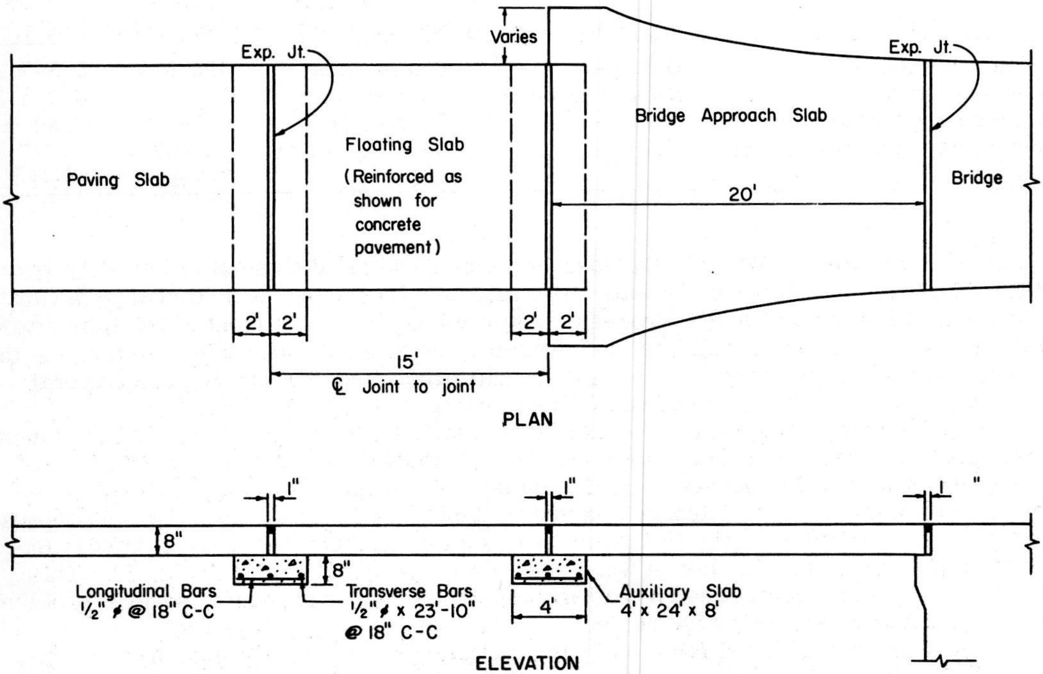

Figure 3. Expansion joint detail (Falls-McLennan County).

that were distinctly different. For comparative purposes an average crack spacing was taken for each of several sections. Using the concrete field laboratory reports for this project as the basis, a study was made to isolate some of the factors that affect the crack pattern. The study resulted in a light of suspicion being cast on numerous factors; but age, concrete strength, curing temperature, and presence of an adjacent lane were found to be definite factors that influenced the crack pattern. The comparison was made on the basis of sections of the project 350-500 ft long where all but one of the various factors that might influence the crack pattern were of fairly uniform value.

Age. —This factor has been pointed out numerous times by reports on this type of pavement from other states. An attempt was made in this investigation to study the variation in the average crack spacing during the first 200 days after placement of the pavement. The presence of the dark curing compound on the pavement during the first week seriously curtailed observations. Even after the compound bleached out, the cracks (being very tight) were extremely hard to locate. Note the difficulty in finding the crack beneath the pencil in Figure 5. What appears to be a crack to the left is a rough place on the edge caused by a crack in the forms. This picture was taken when the pavement was 10 days old. As the age of the pavement increases, the surface width of the crack becomes progressively wider and easier to find (Fig. 6), but observations at the edge of the slab revealed the cracks to be tightly closed immediately below the surface.

Because a day to day survey was not maintained on a specific section, a special approach was used to find the effect of age. The first survey conducted included sections of pavement from 8 to 76 days old. These sections were resurveyed at the age of 200 days. For comparative purposes the ratio of the average crack spacing at the time of the first survey to that observed at the age of 200 days, expressed as a percentage, was plotted against the age of the slab at the time of the first survey (Fig. 7). This relation indicates that the crack development occurs largely during the first 30 days after placement of the slab. With this in mind, a logical deduction would be that the major portion of the crack pattern has formed before the concrete attains any large percentage of its strength. Therefore, a light of apprehension is cast on the practice of relying solely on the ratio of the strength of concrete to the strength of steel for determining the percent steel required.

Concrete Strength. —As pointed out previously field tests were not run on the tensile strength of concrete, but the Texas Highway Department specifications do require 7-day flexural strength determinations at frequent intervals. These values were used as the basis for determining the effect of concrete strength on the average crack

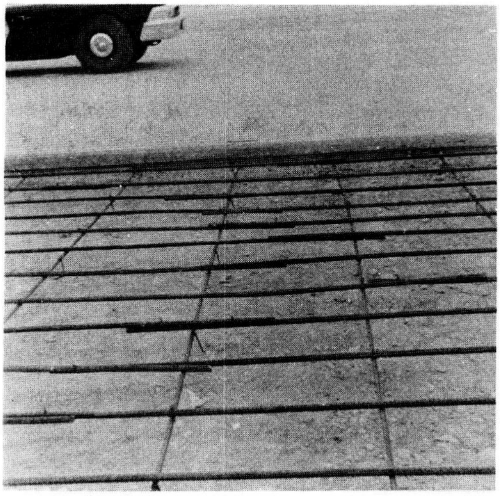

Figure 4. Assembled steel for an acceleration lane ready for concrete pouring operations (Falls-McLennan County Project).

Figure 5. Edge view of typical crack at the age of 10 days (Falls-McLennan County Project).

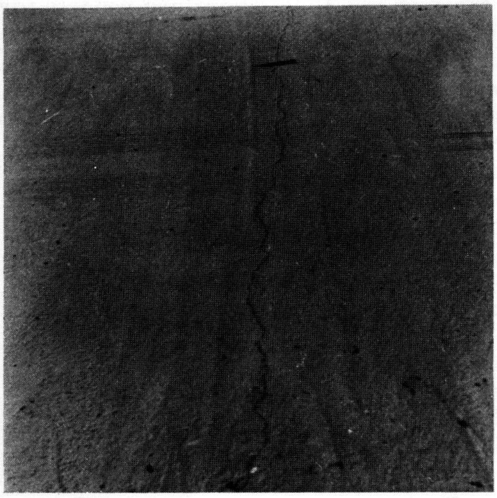

Figure 6. Typical crack at the age of 100 days (Falls-McLennan County Project).

spacing, because the tensile strength is proportional to the flexural strength.

Figure 8 shows the relation between the average crack spacing and the average 7-day flexural strength for this project. The variation in flexural strengths within a selected section was not in excess of ± 20 psi, therefore, the average obtained is felt to be representative of the actual conditions existing. Although the amount of data is limited after the various factors are isolated, the plot does point out that the average crack spacing is directly proportional to the concrete strength. An examination of Vetter's theoretical formula for the crack spacing discloses the crack spacing to be directly proportional to the second order of the concrete tensile strength (3). The data is too limited to make a positive statement as to whether these factors are proportional to the first order as shown or to a higher order, but it may be concluded that the concrete strength has a definite effect upon the crack spacing.

This experience points out the need for a maximum allowable concrete strength if an optimum average crack spacing is to be maintained. Excessive concrete flexural

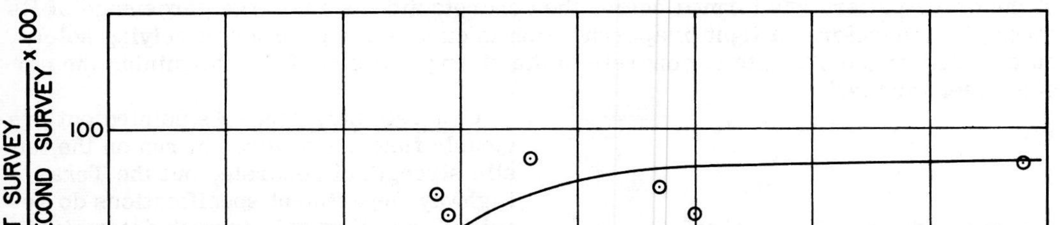

Figure 7. Effect of age on the average crack spacing (Falls-McLennan County Project).

strengths will result in crack spacings and crack widths that cannot be tolerated if the continuity in a continuously-reinforced pavement is to be attained.

Two separate lines representing two curing temperature ranges are depicted in Figure 8. The wide variation in crack spacing for equal concrete flexural strengths led to an investigation into the effect of curing temperature on the crack spacing.

Curing Temperature. —The extended period of paving operations from January to June resulted in a large variation in curing temperature on sections of the pavement over the project. For analyzing purposes, only those sections placed before noon were used. The maximum air temperature experienced by the pavement during the first 8 hr after placement was selected as the curing temperature. As a further precaution, sections experiencing an extreme temperature change in the first 7 days were eliminated.

Figure 9 reveals that the crack spacing is inversely proportional to the curing air temperature of the concrete. The effect of curing temperature on crack spacing is rather pronounced. Note that an air temperature drop of 20 F will result in an increase of 8 ft in the crack spacing. This trend should be expected, inasmuch as a higher air temperature during curing results in a larger amplitude of shrinkage for an equal period of time. Therefore, higher shrinkage stresses during the period when the concrete is relatively weak increases the frequency of cracking. Inasmuch as this pavement has not experienced the effect of long periods of cold weather, it would be rather stigmatic to place too much emphasis on this relation at the present time. If this spread is retained after several seasonal changes, the expected curing temperature of the concrete should be evaluated as a prime design consideration for this type of pavement.

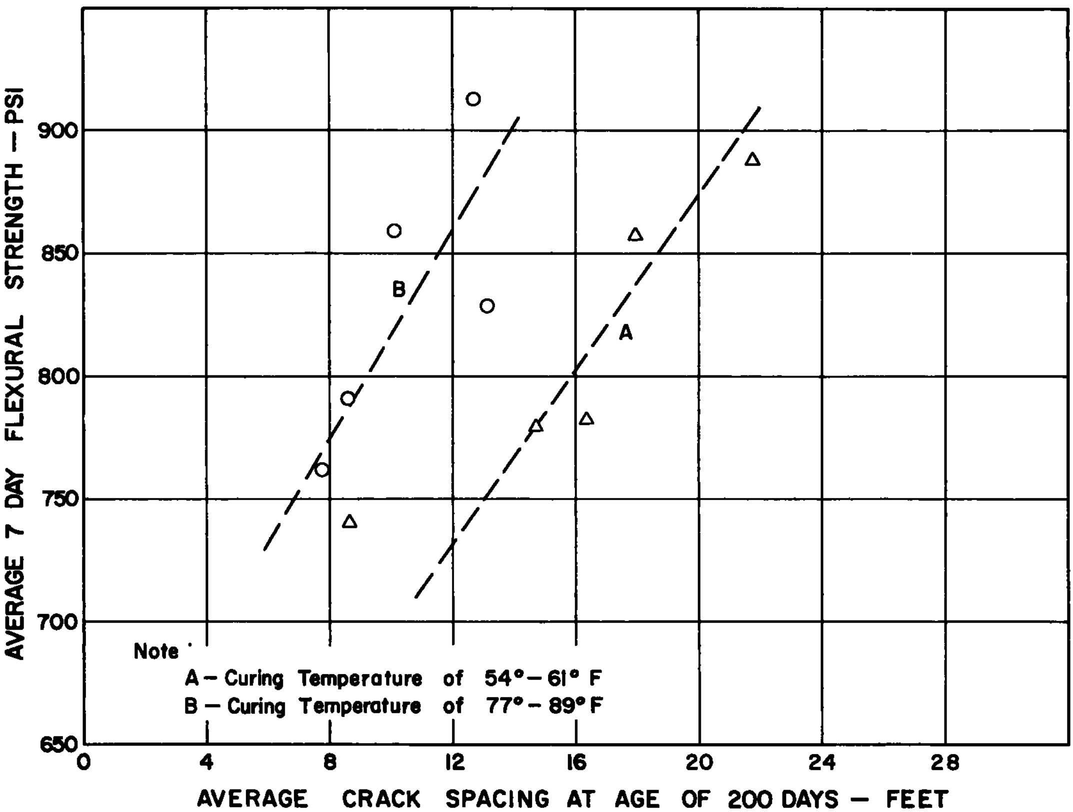

Figure 8. Relation between average crack spacing and average 7-day flexural strength (Falls-McLennan County Project).

Adjacent Lane. —At various locations over the project speed change lanes were poured adjacent to the main lanes. These additional lanes were always poured at a later date, but the time interval between pours was not constant. With this layout, a study was made to determine what effect an adjacent lane had on the crack pattern.

The data revealed that the crack spacing of the older section was not affected by the later pour. Taking the other case, it was found that the lane with the earliest date of pouring affected the crack spacing on the lane with the succeeding pour in every instance. The degree of influence was found to vary inversely with the time lag between pours. For a difference in age of 2 days, 20 percent of the cracks in the speed change lane were apparent continuations of cracks in the main lane. This percentage reduced to 10 percent at an age difference of 30 days. The cracking pattern indicated that these crack continuations in the youngest pavement originated at the common face.

This common cracking is apparently the result of the friction between the two slabs along the vertical face of the longitudinal construction joint. The slight movement in the vicinity of the crack in the older slab is resisted by the friction of the newer slab at the longitudinal construction joint. This results in a quick buildup of stress in the adjacent slab. The newer slab which is still relatively weak cracks at the face due to this added localized stress concentration. The crack then continues to progress across the slab, because the transverse cross-sectional area—the area resisting the stresses induced by volumetric change—is reduced as the crack progresses. These observations further indicate that the major part of the cracking in a continuously-reinforced pavement occurs in the first few days after placement. A surmise on the basis of data indicates that friction, whether it is from another slab or from the subgrade, provides added resistance to the volumetric change of the concrete.

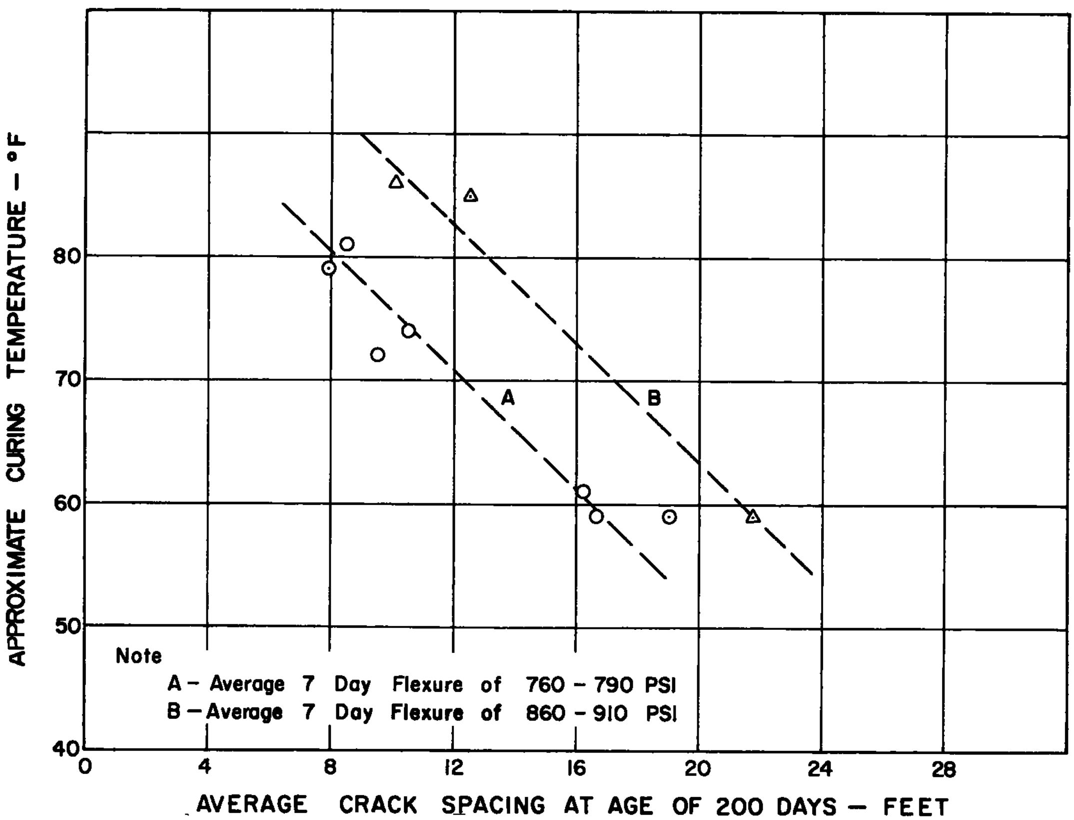

Figure 9. Relation between average crack spacing and air temperature at the time of curing (Falls-McLennan County Project).

Another interesting feature of this study was the presence of a crack in the adjacent slab opposite almost every transverse construction joint on the project where an additional lane was located. The crack was present regardless of which slab was poured first or of the age difference of the two. This indicates, as expected, a greater movement is taking place at the transverse construction joint than at a normal crack.

TARRANT COUNTY PROJECTS

General

The Tarrant County Projects (Numbers 1 through 4, Table 3) were the first continuously-reinforced concrete pavements constructed in Texas. These projects constitute five separate contracts totaling 5.9 mi of expressway-type construction in the city limits of Fort Worth. Of this total, 1.3 mi are on the North-South Expressway and the remaining 4.6 mi are on the East-West Expressway. Slab lengths between expansion joints ranged from 570 ft to 7,681 ft (Table 1). Expansion joints were placed at the ends of structures only. These projects called for 0.7 percent of longitudinal steel. The first project was constructed in 1951 and the fourth and last project using this design was constructed in the latter part of 1957. Continuously-reinforced pavement was used on the main lanes of these projects. A standard jointed reinforced pavement was used on the frontage roads. These pavements are located in an area of low rolling hills. There are a few moderate fills, but principally the main lanes are of the depressed freeway type of construction. The subgrade soils throughout this area are fairly uniform, consisting of a potentially pumping silty clay.

Cross-Section Description

The continuously-reinforced portions of the roadways are 22 ft and 34 ft wide. Reinforced curb and gutter sections with joints at 30-ft intervals are attached to both edges of the pavement by means of a tongue and groove construction joint in combination with ½-in. round tie bars at 2-ft centers. The curb and gutter were poured monolithically, the entire assemblies being 1 and 2 ft wide with 0.5-ft curbs. The outside gutter is 1.5 ft wide and the inside gutter is 0.5 ft wide bringing the total roadway width to 24 ft and 36 ft, respectively. Figure 10 is a typical cross-section for these projects.

A uniform thickness of 8 in. was used for these pavements. The thickness of granular subbase ranged from 4 to 8 in. depending on sub-soil conditions.

Design and Construction Procedures

The concrete for this project was mixed and placed in accordance with the standard Texas Highway Department specifications, Item 320, for concrete pavement (2). Table 2 gives a comparison of specification requirements to the actual values obtained in the field. Although the data show that the range from the minimum to the maximum flexural strength is 175 psi, investigations showed that over 90 percent of the flexural strengths fell within a 125-psi range.

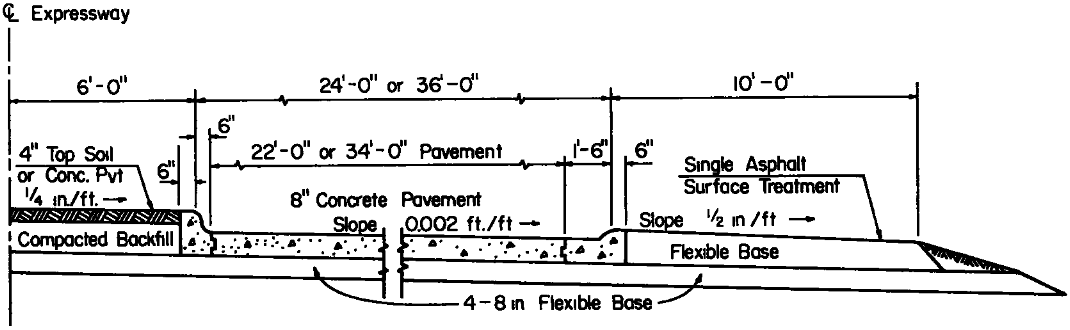

Figure 10. Typical half section for Tarrant County Projects.

The pavement was placed in 22-ft widths on the two-lane sections, and by a combination of 23.5- and 10.5-ft widths for three-lane sections. The two were joined together by means of a tongue and groove longitudinal construction joint in combination with ½-in. round tie bars at 2-ft centers. A formed longitudinal hinged joint was placed in the center of the two-lane placements. No additional steel was used at the hinged joint.

Expansion joints with auxiliary slabs were installed at the bridge ends on these projects in most cases. The joint filler was two ¾-in. boards with top sealed. A joint was placed at each end of a floating slab which was placed between the bridge approach slab and the pavement end. Finger-type bridge expansion joints were installed at a few locations.

The longitudinal steel consisted of ¾-in. round bars placed at 8-in. centers, and the transverse steel consisted of ½-in. round bars at 2-ft centers. The specifications required the use of hard grade steel for all longitudinal reinforcement. All bars were lapped 20 diameters and tied. The steel was assembled in place, and the concrete placement was made for the total depth with the steel being held in the proper position by means of bar chairs spaced at 24-in. centers along alternate transverse bars.

One of the projects (Project 2, Table 3) carried traffic on the subbase course for a period of time before concrete paving operations were started. A single asphalt surface treatment was applied to the 8-in. subbase course, and served as the subbase grade for the concrete pavement.

Performance of the Pavement

These pavements with 0.7 percent steel range from 2 to 8 yr old. The traffic count for the East-West Expressway and the North-South Expressway is 15,700 and 12,300 vehicles per day, respectively. The pavements have been under sporadic observation since their completion. However, an extensive crack survey was not made until this year; therefore, no data can be presented in relation to the effect of age on the crack spacing.

There is an extensive cracking of the pavement, and the cracks are quite visible at any season during the year. The surface width of the crack appears to be appreciable, but immediately below the surface the crack is held tightly closed (Fig. 11). The exaggerated width of the crack at the surface is a result of the concrete wearing which is attributed to the type of aggregate used. Note how fine the crack is at the edge of the slab and worn wide in the wheel paths. It is impossible to detect the cracks when riding in an automobile at a normal rate of speed from the standpoint of visibility or riding quality.

The pavements are in excellent shape and have been entirely free of maintenance. Spalling has occurred at only two cracks on the projects. Figure 12 portrays one of the cracks at which spalling has occurred and reveals that it is not of a serious nature.

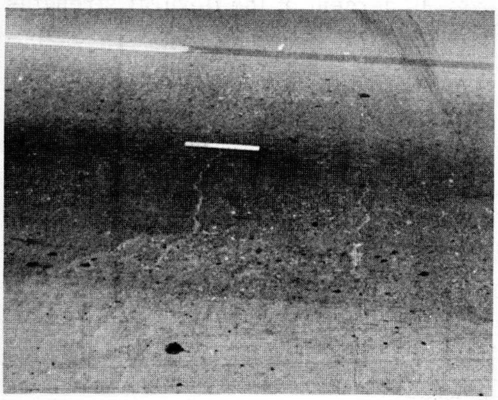

Figure 11. Typical transverse crack at age of 8 years (Tarrant County Project).

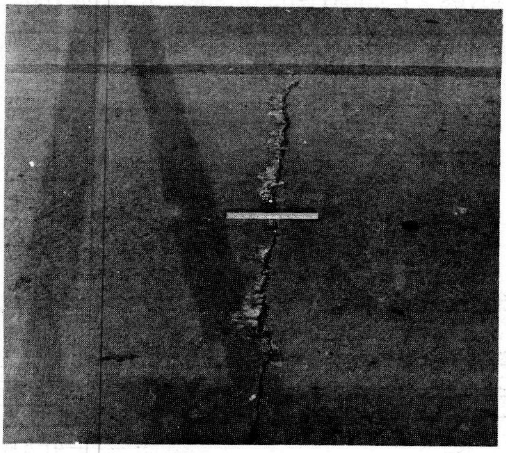

Figure 12. Spalling at transverse crack (Tarrant County Project).

At one location about 50 ft of the pavement has experienced aggregate pop-out, but this is attributed to the type of coarse aggregate used. On the North-South Expressway a drainage problem existed, and the pavement was inundated for a considerable period of time. Observations at this location revealed a normal crack pattern with no sign of failure.

The expansion joints at the structures and the construction joints are entirely free of distress. Under normal circumstances of construction, the transverse construction joints fit into the crack pattern and do not materially affect it. The exception to this generalization is at locations where there was a delay of 3 days or more between pouring the slab on each side of the construction joint. The longitudinal hinged joint and the longitudinal construction joints are open, but both are tighter at the surface than the transverse joints. Observations at the expansion joints indicate that the ends of the pavement have experienced both expansive and contractive movement.

Crack Spacing

A detailed crack survey was made on these projects for the purpose of determining, if possible, what factors influence the crack pattern. The average crack spacing for these pavements involved was found to be 3.7 ft, but this value has little meaning when the variables within each short section of the pavement are considered. Sufficient data were available for evaluating the effect of length of slab, the concrete strength, and the effect of an adjacent lane on the crack pattern. Inasmuch as the concrete on all the Fort Worth projects was placed in the summer season, the effect of curing temperature after several seasonal changes could not be evaluated due to the limited temperature differential.

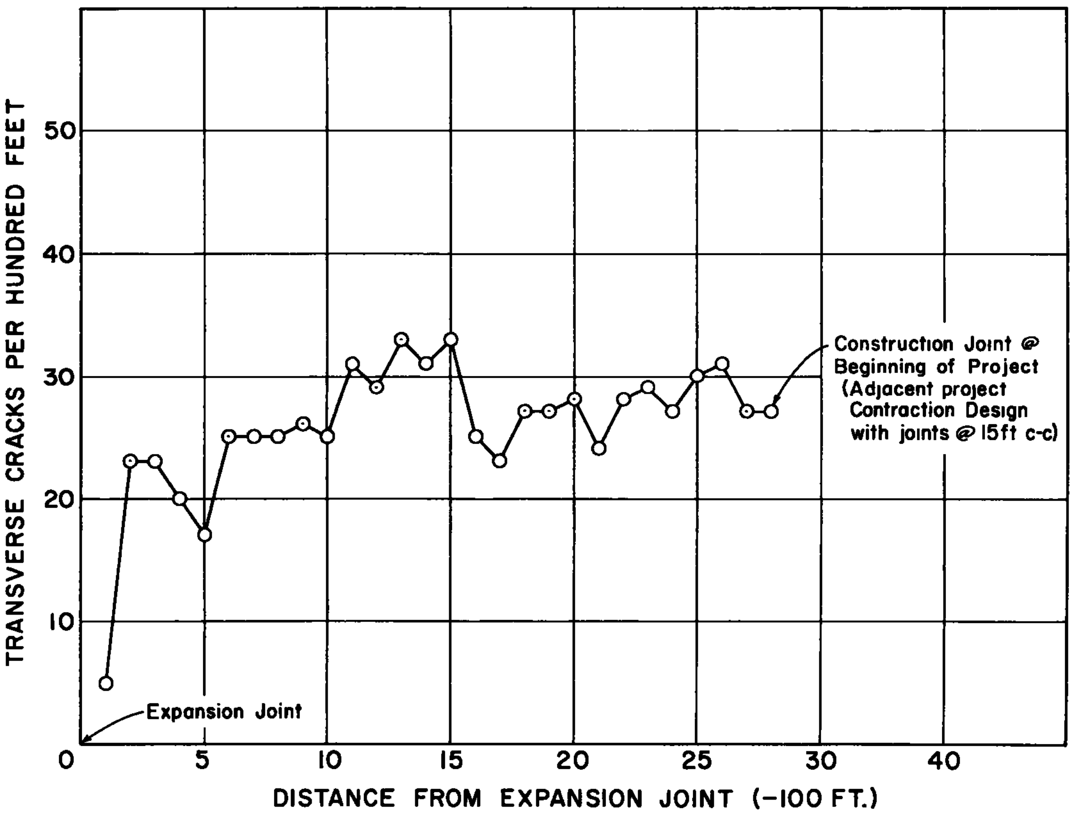

Figure 13. Frequency distribution of transverse cracks (Tarrant County Project).

Length of Slab. —A study was made on these projects to determine the variation in crack frequency between the terminals of each continuous slab. Surveys in other states have shown that the frequency of cracks increases fairly uniformly until a maximum is reached at some interval away from the end of the slab. Then the frequency decreases for a short distance, after which a fairly uniform frequency in the center portion of the slab is attained. Figure 13 is a similar analysis of a typical Tarrant County section. This type of analysis revealed a spasmodic crack pattern with a complete absence of symmetry. The pattern in relation to symmetry remained erratic regardless of the length of section selected for analysis.

The analysis did point out one fact. It was found that a crack pattern was obtained, even though erratic, at a distance of approximately 150 ft from the expansion joint. This indicates—as has been found in other states—that the end portions of the pavement are moving over the subgrade. This distance of 150 ft corresponds to the length of slab end expected to move over the subgrade after evaluating the end section in accordance with the "subgrade drag" approach using a friction factor of two.

Concrete Strength. —When the results of the 7-day flexural strengths were plotted in the proper locations on the foregoing analyses, the crack pattern became more comprehensible. In sections where the flexural strengths were relatively low the frequency of the crack spacing increased in respect to areas of higher flexural strength.

Long sections of pavement with fairly uniform flexural strengths were not present on these projects. Therefore, the average crack spacing of short sections 30 to 100 ft long in the vicinity of the test beams were used for comparison. Due to the small crack spacings involved, the average crack spacing of these short distances presented a representative average.

The relationship between the 7-day flexural strength and the average crack spacing

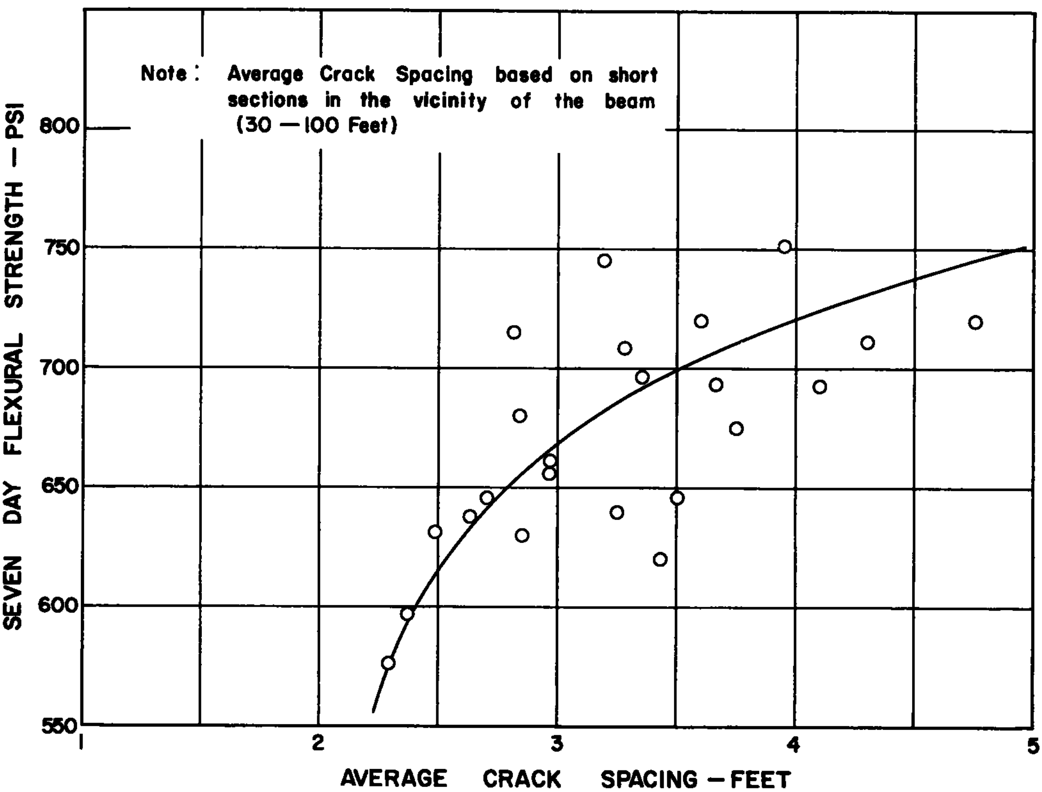

Figure 14. Relation between average crack spacing and average 7-day flexural strength (Tarrant County Project).

for these projects is shown in Figure 14. The figure reveals that these factors are directly proportional to one another. This is the same conclusion deducted from the same type of study on the Falls-McLennan County Project. A second order curve gives a better coverage of points than the linear relation used with the analysis on the other project. This second order relation compares favorably with the effect of tensile strength obtained from Vetter's proposed analysis of crack spacing (3).

A study of the curve shows that an increase of 150 psi in flexural strength will result in an increase of about 2.5 ft in the average crack spacing. This is a rather marked change from the standpoint of the small crack interval on these projects. The pronounced effect of concrete strength on the crack spacing emphasizes the need for uniformity in the concrete strength.

A further look at the curve indicates an average crack spacing of approximately 3 ft for a flexural strength of 650 psi. The concrete for these projects was designed with the intention of providing the aforementioned strength. This value for crack spacing is about one-half of the maximum allowable crack spacing for maintaining the continuity of the pavement. Based on this evidence it might logically be concluded that the amount of steel used on this project is in excess of the minimum amount required for an optimum crack spacing considering the values of the other factors.

Adjacent Lane. —The presence of adjacent placements 23.5 and 10.5 ft wide provided data for a comparative study of the effect of adjacent placements on the crack pattern. Inasmuch as the traffic distribution over the three lanes is fairly uniform, this factor is assumed to be a static one.

A study of the crack patterns of the adjacent lanes on these projects indicates the slab which is poured first has a marked influence on the crack pattern of the second slab. Approximately 60 percent of the cracks in the youngest slab were found to be continuations of cracks in the slab with the earliest date of pour. The average time lag between pours on the adjacent slabs was 7 days. This is a considerable differential in results from those found on the Falls-McLennan County Project.

PRESENT DESIGN PROCEDURE FOR CONTINUOUSLY-REINFORCED PAVEMENT

The design of a continuously-reinforced concrete pavement is extremely difficult due to the many variables encountered. The effect of a number of variables on continuously-reinforced pavements in Texas is evaluated in the preceding sections, but this is only a small portion of the variables. Among the known variables which should be considered are: tensile, bond, and flexural strength of the concrete; tensile and shear strength of the steel; steel bond area; temperature differential expected; shrinkage coefficient of the concrete; combined modulus of elasticity of the concrete and steel; modulus of subgrade reaction (k) developed; resistance to fatigue of the combined concrete and steel; and intensity, duration, frequency of application, and lateral placement of loads. Inability to properly evaluate and correlate each of these values with respect to each other makes a theoretical approach to design apparently beyond grasp. Therefore, it is proposed that the design of this type of pavement be based on a combination of theory and experience along with experimental extrapolation.

At the present time these variables are evaluated, insofar as possible, in three basic design steps, these being: (1) the pavement thickness determined by an interior loading condition; (2) the use of sufficient steel to adequately control and restrain volume changes caused by shrinkage and temperature differential; and (3) the use of additional steel at the edge to compensate for the difference in thickness between interior and edge loading conditions.

Pavement Thickness

The pavement thickness is determined by Westergaard's original formula for an interior loading condition (4). This approach is used because a majority of the modifications of this original formula are based on experiments with jointed pavement. It is recognized that due to the non-homogeneous nature of a continuously-reinforced concrete slab, it may be argued that Westergaard's theoretical approach does not apply.

However, close observations of this pavement design under traffic loads indicate an extremely high degree of load transfer efficiency exists at the cracks lending credence to its use. The pavement thickness determined by this formula is used uniformly across the width of the slab. The variables contained in this formula are familiar to most designers, however, special emphasis might be given here to the methods used to evaluate the modulus of rupture, modulus of elasticity, and wheel load.

Modulus of Rupture. —The specifications generally require that the concrete be designed with the intention of producing a modulus of rupture—flexural strength—of 600 psi at the age of 7 days. For design purposes the 7-day modulus of rupture is divided by two. The Maryland Road Test (5) and research by the Portland Cement Association (6) shows this safety factor will be adequate to prevent excessive fatigue. In some instances the choice of a suitable modulus of rupture will depend on available materials and local conditions.

Modulus of Elasticity. —For the aforementioned specified flexural strength, a modulus of elasticity of 4,000,000 psi is used. In cases where different flexural strengths are required, the tangent modulus of elasticity is determined for a test batch of concrete that has been prepared with the actual materials from which the final concrete will be made.

Wheel Load. —The Texas Highway Department uses the average of the ten heaviest daily wheel loads expected on the portion of the highway system under consideration as the basis of design for rigid-type pavements. Where continuously-reinforced concrete pavements were considered for use, a study of loadometer data indicated a wheel load of 16,000 lb would be adequate. This figure encompasses the predicted wheel load increase and effect of tandem axles expected during the life of the facility.

Using the foregoing considerations for modulus of rupture, modulus of elasticity, wheel load, and a modulus of subgrade reaction (k) of 100 in Westergaard's interior formula, a pavement thickness of 7½ in. was arrived at. This figure was rounded off to 8 in. for use in construction.

Controlling Volume Change

Steel is placed in the pavement for the purpose of controlling and restraining the volume changes caused by hydration shrinkage of the concrete and a change in temperature of the concrete. Positive temperature differentials (that is, increases in temperature from final curing temperature) present no problem in design because plain concrete is capable of taking the compressive stresses developed by such temperature increases. Shrinkage along with a negative temperature differential results in tensile stresses in the pavement. When the tensile stress in the concrete exceeds its tensile strength, a crack is formed.

The optimum economical design for the steel used in a continuously-reinforced concrete pavement would be one with a balanced combination of concrete properties and steel properties such as to provide a crack width that will prevent pavement deterioration. If the crack width of the uncontrolled cracking—which is characteristic of a continuously-reinforced pavement—attains sufficient magnitude, the crack in effect becomes an uncontrolled joint which will lead to progressive failure of the pavement. Although several theoretical formulas have been presented in the past to evaluate these factors, none appear to be all encompassing. Therefore, the current steel design details used for this type of pavement are the result of theoretical approaches and years of experience.

Crack Width. —To prevent pavement deterioration the cracks in the pavement must be of small enough magnitude to permit the retention of sufficient granular interlock and prevent the entrance of water. The load transfer from slab unit—the area between the cracks—to another slab unit is dependent on granular interlock. Based on experience with contraction joints and a limited amount of laboratory tests, a crack width of 0.02 in. is felt to be the absolute maximum allowable for structural integrity. Cracks wider than this disrupt the continuity of the pavement and permit rapid deterioration.

Inasmuch as crack width has been found to be directly proportional to the crack spacing in a linear relation (7, 8), a logical approach would be to design from the standpoint of crack spacing.

Crack Spacing. —An optimum crack spacing would be one which insures that the permissible crack width is not exceeded during the severest road conditions. Although the conclusion is reached from a surmising standpoint, it is felt that the design should strive for an optimum crack spacing of 5 to 6 ft. As stated previously this optimum crack spacing, or crack width, is dependent on a proper correlation of the steel properties and of the concrete properties.

Steel. —To be justifiable from an economical standpoint only sufficient steel to accomplish the aforementioned desired results should be placed in a continuously-reinforced pavement. Because this type of pavement is free to move in the transverse direction and is relatively fixed at the ends in the longitudinal direction, two different methods of design are employed.

The "subgrade drag" theory is used to determine the amount of steel required in the transverse direction. The current design details for the placement of the steel provides for 0.1 percent intermediate grade steel in the transverse direction. This provides adequate steel for a three-lane pavement.

Providing for the proper amount of steel in the longitudinal direction presents a more complicated problem. The steel percentage for the first continuous pavement in Texas was based on the theory of providing ample steel to insure that the tensile strength of the steel exceeded the tensile strength of the concrete. Based on this assumption 0.7 percent hard grade steel was required for the concrete strengths expected in the area.

As pointed out previously, the average crack spacing for these pavements was from 3.0 to 4.0 ft. The pavement maintained excellent continuity, and no deterioration was noted at the cracks. This crack spacing was excellent from a design continuity standpoint, but from an economic standpoint it indicated an excessive amount of steel was used. It is believed that three apparent fallacies existed in the approach used for this design.

One is neglect of the frictional resistance to volume change by the subgrade material. An examination of cracks in continuously-reinforced pavement in Texas and other states shows that the crack becomes progressively narrow from the top to the bottom of the slab for mid-depth steel. A plausible explanation of this phenomenon is that the subgrade or subbase is resisting a portion of the volumetric change. An intelligent evaluation of this phenomenon presents an imposing problem. Another factor that would tend to lower the stresses and reduce the amount of required steel would be the plastic flow in the concrete. Although many investigators have pointed out this latter point, a feasible evaluation of this factor has not been presented. Another pertinent factor which would lower the percentage of steel is the bond-shrinkage relation. Investigators have pointed out that the bond strength of the concrete increases much faster than the flexural, compressive, or tensile strength. One investigator concluded that the bond between the concrete and steel is largely a result of the shrinkage of the concrete on to the steel and it would have the same time relation as shrinkage (9). Inasmuch as the bond between the steel and the concrete is a resisting factor to volumetric change, tensile stresses developed in the concrete will increase much faster than the tensile strength of the concrete. Therefore, the concrete cracks at a much lower stress than the one used for design purposes. Hence, a lower tensile strength may be used for design purposes—this results in a smaller percentage of steel.

Due to the inability to definitely evaluate these factors experience must be relied on to set the percentage of steel at a safe economical value. Experience in other states as well as in Texas seems to point out that 0.5 percent steel is a safe minimum to use. In addition it is believed that by controlling the maximum strength of the concrete within a workable tolerance, it is reasonable to allow the use of intermediate grade steel. Using this percentage as a basis the Texas Highway Department developed a recommended set of design details for continuously-reinforced concrete pavement. The original standard for bar reinforcement provided for three different bar sizes and spacings to be placed at mid-depth, these being ¾ in. ϕ at 11 in. c-c, ⅝ in. ϕ at 7.5 in. c-c, and ½ in. ϕ at 5 in. c-c. Although the pounds per square yard of steel differed slightly, these variations were felt to be compatible alternates for bars. In addition to the detail for bar reinforcement an alternate detail using welded wire fabric was prepared.

A limited investigation pointed out the feasibility of lowering the required steel percentage to 0.4 for this material. This takes into consideration the higher yield point stress of welded wire fabric.

A rather extensive study of available data on crack spacing in other states and data in Texas revealed that equal percentages of steel did not result in equal crack spacing in all instances. By holding as many variables as possible constant— such as season of year concrete poured, beam strength, etc.—it was found that different bar sizes for comparable percentages of steel had dissimilar crack patterns. Therefore, crack spacing was compared to the bond area of steel per volume of concrete. This comparison disclosed that the crack spacing is inversely proportional to the bond area up to a point where the bond area is sufficient to develop the yield strength of the steel.

Using this relation the design details for the bars were re-examined. The ¾-in. bar at 11-in. centers was found to be deficient in bond area for the optimum crack spacing. Hence, this combination was deleted from the design detail. Because welded wire fabric depends largely on anchorage rather than bond for stress development, the bond area per volume of concrete approach could not be used to evaluate the varying wire sizes for the fabric.

Concrete Strength.—As pointed out previously the Texas Highway Department controls strength requirements for concrete through 7-day tests for flexural strength. Consequently the tensile strength of concrete may be controlled indirectly through flexural strength inasmuch as tensile strength and flexural strength are interrelated. An inspection of field laboratory reports for concrete used on the recent continuously-reinforced project in Falls and McLennan Counties revealed that 7-day flexural strengths in the range of 650 to 920 psi were being obtained under normal construction procedures.

A review of the basic formula for determining the percent steel by a ratio of the tensile strength of concrete and steel shows the percent of steel required is directly proportional to the tensile strength of the concrete. The tensile strengths obtained on the Falls-McLennan County Project were possibly in the danger zone for overstressing the steel. To guard against the reoccurrence of the foregoing situation and in direct conflict with the old adage "the stronger the concrete the better," the need for a maximum strength requirement became obvious. The present recommended design procedure is to base the slab thickness on a 7-day flexural strength of 600 psi using a safety factor of two. Enough flexural strength must be retained to resist the external stresses induced by the wheel loads, therefore, this problem has both upper and lower limits. A range of about 125 psi was the minimum that could be tolerated under present construction practices. A maximum flexural strength of 675 psi and a minimum of 550 psi were resolved to be the feasible limits of flexural strengths for a typical Texas project.

Air entrainment, cement factor, and water-cement ratio are tools that should be used to obtain an optimum mix design that will result in concrete strengths and durability required. Experience in Texas and other areas indicates that an air-entrainment range of 3 to 5 percent will usually result in a satisfactory mix with very little effect on the concrete bond strength (10, 11).

Edge Loading Condition

The difference in required pavement thickness between Westergaard's edge loading equation and interior loading for the conditions prescribed is 2.0 in. The Texas Highway Department at the present time increases the steel area at the edge to compensate for this difference. Although this procedure is considered highly conjectural, it is based on a combination of theory and experience as follows:

1. The theoretical approach is based on the hypothesis that reinforcing steel can carry a percentage of the load in proportion to its equivalent area of concrete.
2. Experience with lightly reinforced jointed concrete pavement where reinforcing steel was added at the edge to compensate for the thickness deficiency.

SUMMARY OF RECENT PROJECTS

Since the first continuously-reinforced concrete pavement was constructed in Tarrant County, a number of other pavements using this principle have been constructed or contracted for construction in Texas. Table 3 and Table 4 list these projects along with other pertinent information. Several of the outstanding features of these projects are discussed in the following paragraphs.

Low Modulus Concrete

Two projects have been contracted in the Fort Worth area which involve the use of continuously-reinforced concrete pavement with a low modulus of elasticity and rupture. These are Project Numbers 13 and 14 as given in Table 3. The first project has been in service since November 1957, and the latter is under construction at the present time.

Because the design and the design factors for both projects are comparable with the exception of the percent steel, this discussion will be limited to Project Number 13. The existing section consisted of 20-ft 9-6-9 concrete pavement which had been surfaced with 2 in. of hot mix asphaltic concrete. This project is in a light industrial section located entirely within the city limits of Fort Worth. The new improvement consisted of widening 22 ft on each side and the addition of curbs for a completed 64-ft street. The new concrete is 7 in. thick, except at intersections where the edge thickness was increased to 10 in., with a 2-in. asphaltic concrete wearing surface.

Continuously-reinforced low modulus concrete was used for widening in this instance for several reasons. The grades in this area were critical because the abutting property had to be closely matched with the curbs. A depth evaluation by the Texas Triaxial Method of design showed that the clayey subgrade soil would require 22 in. of better material for a flexible base design. Inasmuch as the grades were critical and the presence of numerous underground facilities eliminated the possibility of excessive excavation, a rigid type of base which would rest directly on the subgrade was decided on. The presence of a potentially pumping subgrade soil made the elimination of joints highly desirable, hence a continuous-type pavement was selected.

The design of the pavement thickness was based on a modulus of elasticity of 2×10^6 psi, and a modulus of rupture of 450 psi. Table 5 contains a resumé of the field test data for this project. Note the lower coefficient of thermal expansion obtained with the lower cement factor.

At the present time this project presents an extremely smooth riding surface. The pavement has no signs of distress, and maintenance problems have developed at only two locations. One was an elliptical shaped cracking of the pavement at a point where a broken city water line had washed away the subgrade support. The second is the pumping of clear water through a longitudinal construction joint at the south end of the project.

Because the concrete base is covered with asphaltic concrete, a cracking pattern could not be determined. Visible transverse cracks could not be found in the concrete base immediately prior to placing the wearing surface. At the present time reflected cracking is visible only at the longitudinal construction joints and at the joints in the old pavement which are visible in the wearing surface.

Variation of Bond Area

In the first part of 1959 a continuously-reinforced pavement containing 0.5 percent steel was constructed in Tarrant County (Project Number 8, Table 3). A uniform pavement thickness of 6 in. was used on the frontage road, and a uniform thickness of 8 in. was used on the main lanes. Both pavements contained approximately 0.5 percent longitudinal steel. The longitudinal steel consisted of ½-in. round bars at 6.5-in. centers on the 6-in. pavement and ⅝-in. round bars at 7.5-in. centers on the 8-in. pavement. These pavements, therefore, present a wide variation in bond area with the 6-in. pavement having a much greater ratio of bond area to volume of concrete.

Crack surveys were taken on both pavements on this project. An accurate survey

TABLE 3

CONTINUOUSLY REINFORCED CONCRETE PROJECTS CONSTRUCTED IN TEXAS

Ref. No.	County	Highway and Limits	Date Completed	Length Miles	Pavement Thickness (Inches)	Subbase Thickness and Type	% Long. Steel	Longitudinal Bar Size & Spacing	Type of Steel	Comments
1	Tarrant	U.S. 81: Rosedale St. to Morningside Dr.	8/51-10/51	1.301	8	6" Crush. Stone	0.7	3/4" Ø @ 8"	H.G.	
2.	Tarrant	S.H. 550: Camp Bowie Blvd. to Summit Ave.	5/51-9/51	3.438	8	8" Crush. Stone	0.7	3/4" Ø @ 8"	H.G.	
3.	Tarrant	S.H. 550: Wintrop Ave. to Camp Bowie Blvd.	9/51-10/55	0.943	8	4" Crush. Stone	0.7	3/4" Ø @ 8"	H.G.	
4.	Tarrant	S.H. 550: Henderson St. to Lamar	6/56-8/57	0.180	8	8" Crush. Stone	0.7	3/4" Ø @ 8"	H.G.	
5.	Comal	I.H. 35: 0.2 Mi. S. of New Braunfels to S. Bank of Guadalupe River	*	2.381	8	20" Crush. Stone	0.50	3/4" Ø @ 11"		
6.	Harris	I.H. 45: N. Loop Dr. to Airline Dr.	*	1.833	8 ML 6 FR	6" Cement Stab. Bs.	0.55 ML 0.5 FR	5/8" Ø @ 7" 1/2" Ø @ 6.5"	H.G.	Welded wire fabric alternate with 0.46% Longitudinal Steel
7.	Falls & McLennan	I.H. 35: Bell Co. Line to 2.0 Mi. N. of Bruceville	2/59-7/59	6.011	7 & 8	5" Crush. St. 5" Gravel	0.49	3/4" Ø @ 11¼"	H.G.	4000 Ft. of 7" CRCP Overlay Over an existing 9-6-9 Conc. Pavement
8.	Tarrant	I.H. 35: Bellnap Drive to 17th Street	2/59-6/59	1.113	8 ML 6 FR	4" Roadbed Treatment	0.5 ML 0.5 FR	5/8" Ø @ 7.5" 1/2" Ø @ 6.5"	H.G.	
9.	Harris	U.S. 59: Kirby Dr. to Mandell Street	*	0.873	8 ML 6 FR	6" Cement Stab. Bs.	0.55 ML 0.5 FR	5/8" Ø @ 7" 1/2" Ø @ 6.5"	H.G.	Welded Wire Fabric Alternate with 0.46% Longitudinal Steel
10.	Harris	I.H. 610: Fulton St. to Hardy St.	*	0.986	8 ML 6 FR	6" Cement Stab. Bs.	0.55 ML 0.5 FR	5/8" Ø @ 7" 1/2" Ø @ 6.5"	H.G.	Welded Wire Fabric Alternate with 0.46% Longitudinal Steel
11.	Bexar	I.H. 35: Rittman Rd. to Fratt	*	2.629	8	8" Crush. St. 14" Found. Crse.	0.50	3/4" Ø @ 11"		
12.	Kaufman	I.H. 20: Brushy Cr. to Intersection U.S. 80	*	0.949	8	6" Crush.St. 6" Lime Stab.	0.50	**	I.G.	3-5% Air Entr. Req'd. Min. 600 PSI & Max. 700 PSI 7 Day Flex. Str.
13.	Tarrant	U.S. 287: Rosedale St. to Rolling Hills Dr.	11/57	2.111	7	***	0.50	5/8" Ø @ 7.5"	H.G.	Min. 450 PSI 7 Day Flex. Str. Min. 6% Air Entr. Req'd.
14.	Tarrant	U.S. 80: I.H. 820 to Davis St. in Arlington	*	5.998	8	***	0.40	5/8" Ø @ 9.5"	H.G.	Min. 450 PSI 7 Day Flex. Str. Min. 6% Air Entr. Req'd.
15.	Dallas	S.H. 183: I.H. 35E to Trinity River	*	1.316	8	6" Roadbed Treatment	0.50	**	I.G.	3-5% Air Entr. Req'd. Min. 600 PSI & Max. 700 PSI 7 Day Flex. Str.

ML - Main Lane FR - Frontage Road H.G. - Hard Grade I.G. - Intermediate Grade

* Under contract but concrete paving operations have not started.
** A THD Standard used - which permits alternate bar sizes.
*** Concrete pavement serves as base - 2" Asphalt Wearing Surface over concrete pavement.

Note: 1. All steel placed at mid-depth.
2. All projects required a minimum of 650 PSI for 7-day strength unless noted above.

TABLE 4

CONTINUOUSLY REINFORCED CONCRETE PROJECTS*

Ref. No.	County	Highway and Limits	Length Miles	Pavement Thickness (Inches)	Subbase Thickness and Type	% Long. Steel	Longitudinal Bar Size & Spacing	Type of Steel	Comments
16.	Tarrant	I.H. 35W: From 28th St. to 12th St.	0.939	8 ML 6 FR	4" Roadbed Treatment	0.5 ML 0.5 FR	5/8" Ø @ 7.5" 1/2" Ø @ 6.5"	H.G.	6" CRCP to be used on shldrs. also with 2" HMAC
17.	Harris	U.S. 59: Mandell St. to Hawthorne St.	1.664	8 ML 6 FR	6" Cement Stab. Bs.	0.55 ML 0.5 FR	5/8" Ø @ 7" 1/2" Ø @ 6.5"	H.G.	Welded Wire Fabric Alternate with 0.64% Longitudinal Steel
18.	Tarrant	I.H. 35W: 12th St. to Bellnap	1.092	8 ML 6 FR	4" Roadbed Treatment	0.5 ML 0.5 FR	5/8" Ø @ 7.5" 1/2" Ø @ 6.5"	H.G.	
19.	Navarro	U.S. 75: 0.8 Mi. S. of I.H. 45 to 1st. st. in Corsicana	2.535	8	6" Soil - Lime Stab.	0.5	**	I.G.	3-5% Air Req'd.; Min. 600 PSI and Max. 700 PSI, 7 Day
20.	Dallas	I.H. 35E: Bachman-Hines Blvd. to State Highway 14	1.520	8	6" Roadbed Treatment	0.5	**	I.G.	3-5% Air Req'd.; Min. 600 PSI and Max. 700 PSI, 7 Day
21.	Dallas	I.H. 35E: State 14 to Southwell Road	1.542	8	6" Roadbed Treatment	0.5	**	I.G.	3-5% Air Req'd.; Min. 600 PSI and Max. 700 PSI, 7 Day
22.	Walker	I.H. 45: Montgomery County Line to Huntsville Loop	11.348	8	6" Foundation Crse. 6" Lime Stab.	0.5	**	I.G.	3-5% Air Req'd.; Min. 550 PSI and Max. 675 PSI, 7 Day Flex. Str., Slab Ends anchored.
23.	Harris	U.S. 59: Alief Road to McCue Road	1.860	8 ML 6 FR	6" Cement Stab. Base	0.55 ML 0.5 FR	5/8" Ø @ 7" 1/2" Ø @ 6.5"	H.G.	Welded Wire Fabric Alternate with 0.46% Longitudinal Steel
		TOTAL MILEAGE	54.562						

ML - Main Lane FR - Frontage Road H.G. - Hard Grade I.G. - Intermediate Grade

*These plans are in the final stage of preparing for contract.

**A THD Standard used - which permits alternate bar sizes.

NOTE: 1. All steel placed at mid-depth.

2. All projects required a minimum of 650 PSI for 7 Day strength unless noted above.

TABLE 5

SUMMARY OF CONCRETE BASE RESULTS

Item	Units	Minimum	Average	Maximum
Cement factor	Sacks per cu yd	3.2	3.3	3.5
Air entrainment	%	3.8	5.4	6.6
7-day flexural strength	Psi	408	450	492
28-day pure tensile strength	Psi	176	203	236
28-day modulus of elasticity	Psi	1.06×10^6	1.37×10^6	1.69×10^6
Coefficient of expansion	/°F	0.0000032	0.0000039	0.0000047

of cracks was obtained on the main lanes, but the crack pattern on the frontage road was impossible to determine. The cracks on the frontage road were almost invisible due to the hot weather and their tightness. Figure 15 partially illustrates the difficulty in locating a typical crack on the frontage road. Light local traffic had been using the frontage road for a period of time before the photograph was made. Very little trouble was experienced in locating the cracks on the main lane (Fig. 16) even though traffic had not used the main lanes. The pavements will have to experience a winter before a reliable comparison can be made.

Although authentic data was not obtained on this project, the findings do tend to verify the bond area-concrete volume relationship mentioned earlier.

CONCLUSIONS

From the preceding, the following conclusions may be drawn:

A. A final average crack spacing of 6 to 7 ft is believed to be the optimum desired from the standpoint of attaining continuity of the pavement and providing an economical design.

1. The following factors along with age have a definite effect on the crack spacing and should be evaluated in the design:
   a. The percentage of steel.
   b. The bond area available for a given percentage of steel.
   c. The tensile strength of the concrete.
2. The rate of hydration which is affected by the air temperature has an effect on the average crack spacing, and should be considered in the design.

B. An 8-in. slab with a longitudinal steel percentage of 0.5 for intermediate grade steel reinforcement and 0.4 for welded wire fabric is felt to be an adequate design if the bond area of the bars or transverse anchorage of the wire fabric is sufficient.

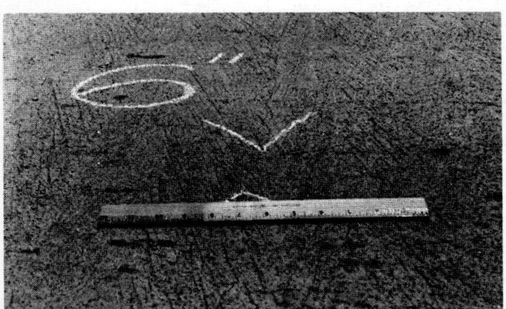

Figure 15. Transverse crack on frontage road, 6-in. pavement (Tarrant County Project).

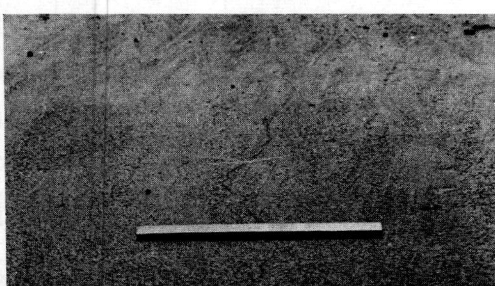

Figure 16. Transverse crack on main lane, 8-in. pavement (Tarrant County Project).

C. A maximum flexural strength of 675 psi and a minimum of 550 psi are felt to be the desired limits of the 7-day flexural strength for continuously-reinforced pavement in Texas.

D. Sufficient steel should be added at transverse construction joints to fully transfer the load across the crack.

E. Because a more satisfactory method than expansion joints is needed for handling the slab ends due to the movement experienced, the Texas Highway Department is experimenting with the use of anchor lugs as a means of solving this problem.

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9. Peattie, K.R., and Pope, J.A., "Effect of Age of Concrete on Bond Resistance." ACI Journal (Feb. 1956).
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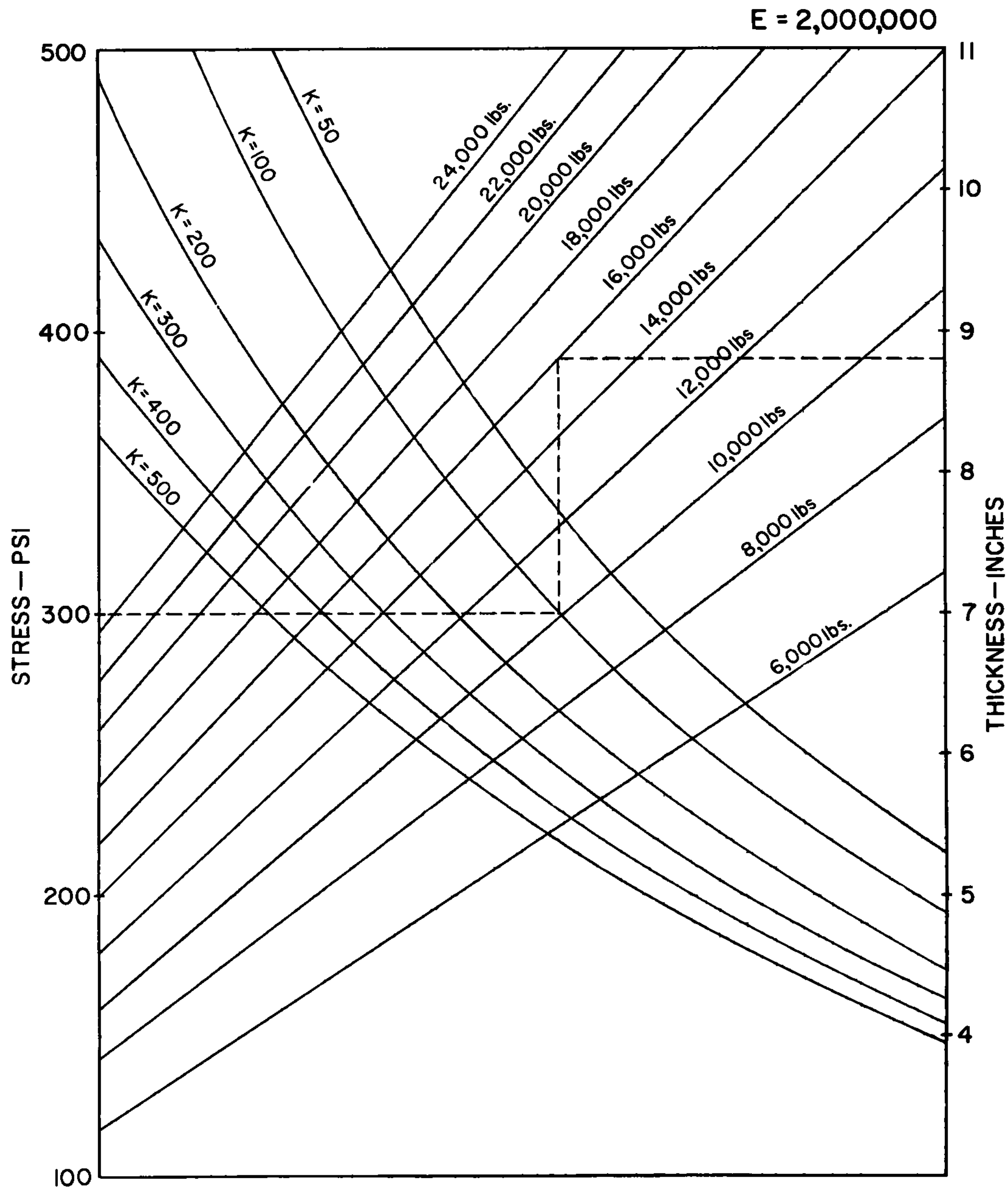
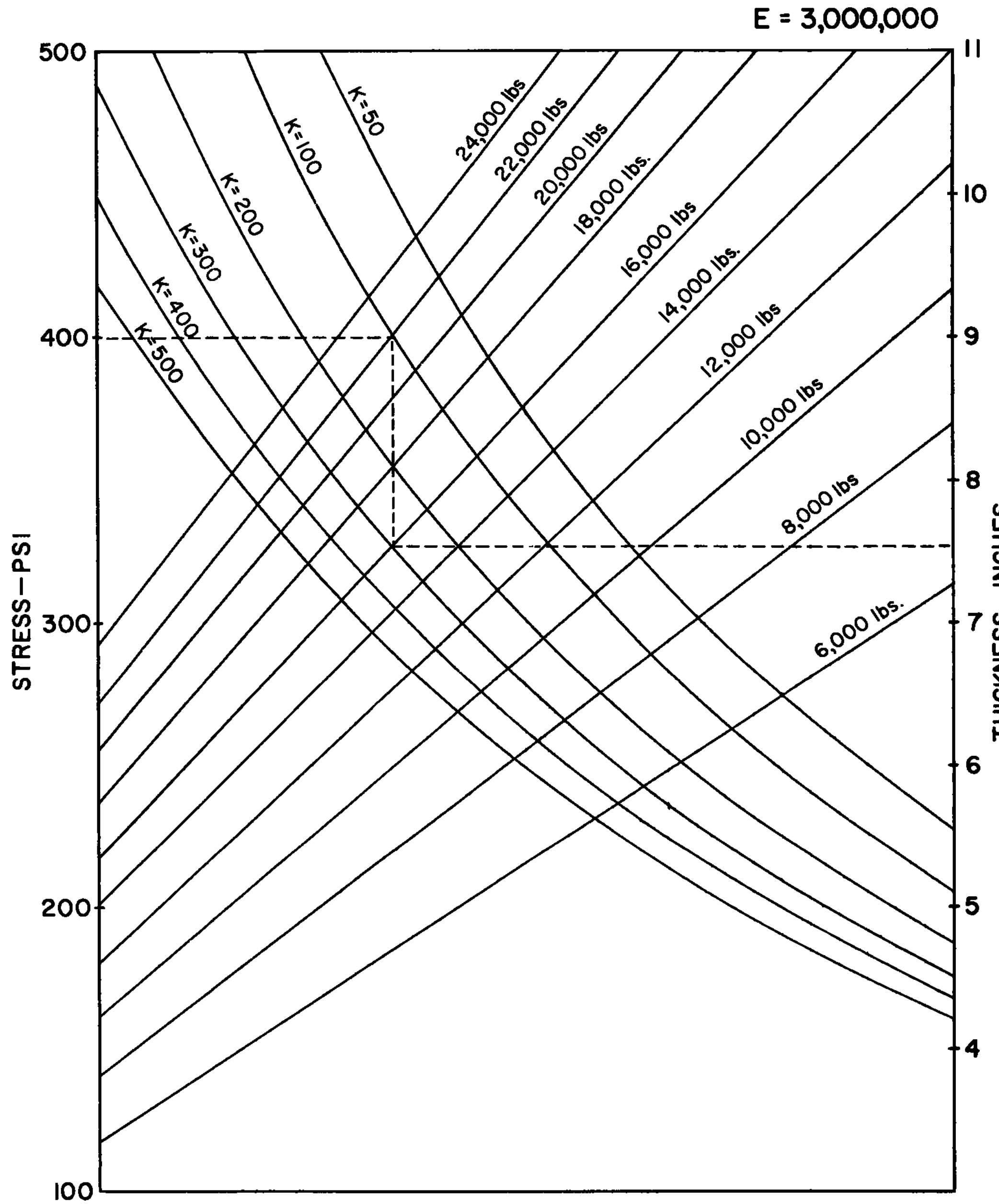
Appendix

DESIGN CHART FOR CONCRETE PAVEMENT
WESTERGAARD'S EDGE LOADING



DESIGN CHART FOR CONCRETE PAVEMENT
WESTERGAARD'S EDGE LOADING



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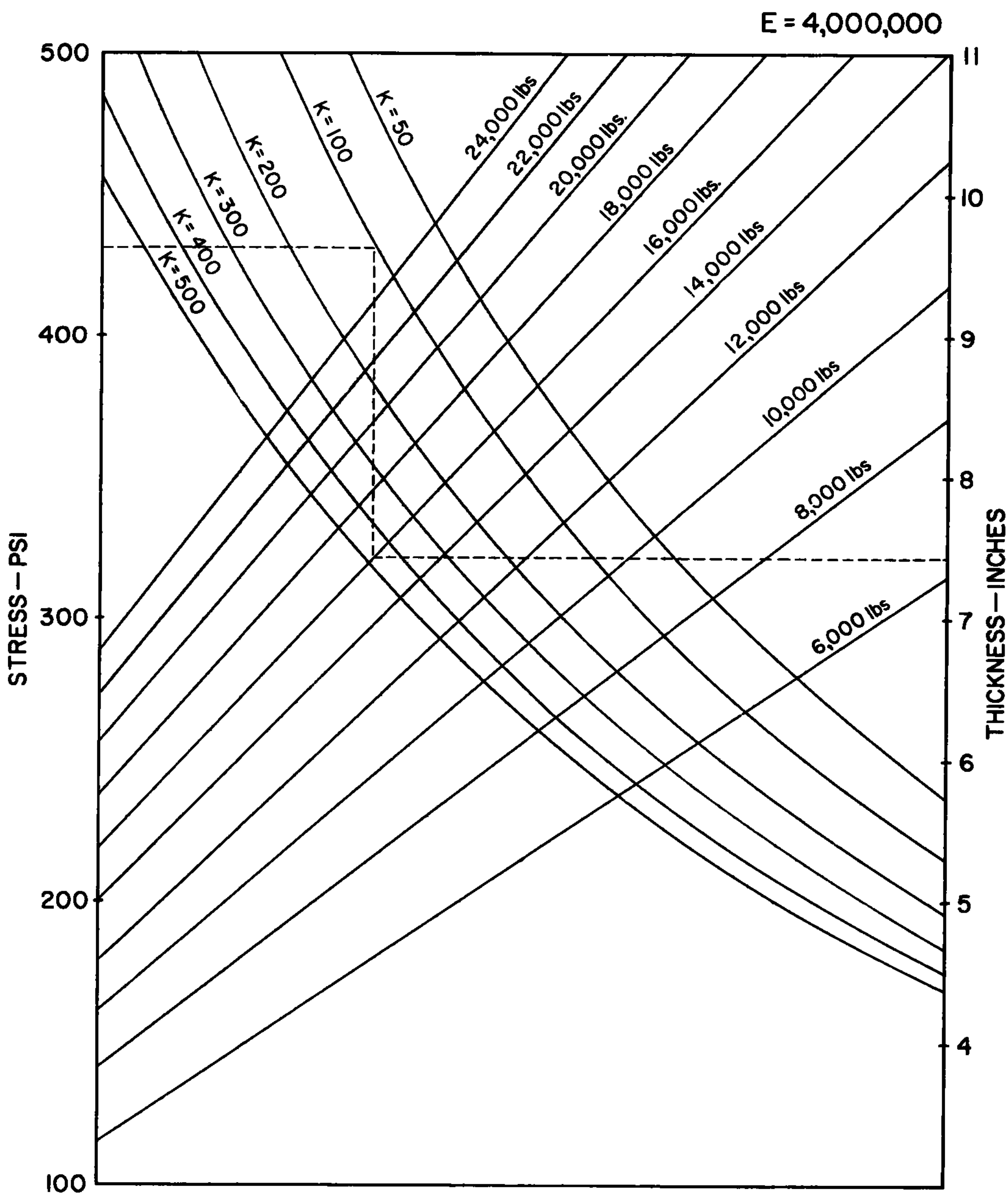
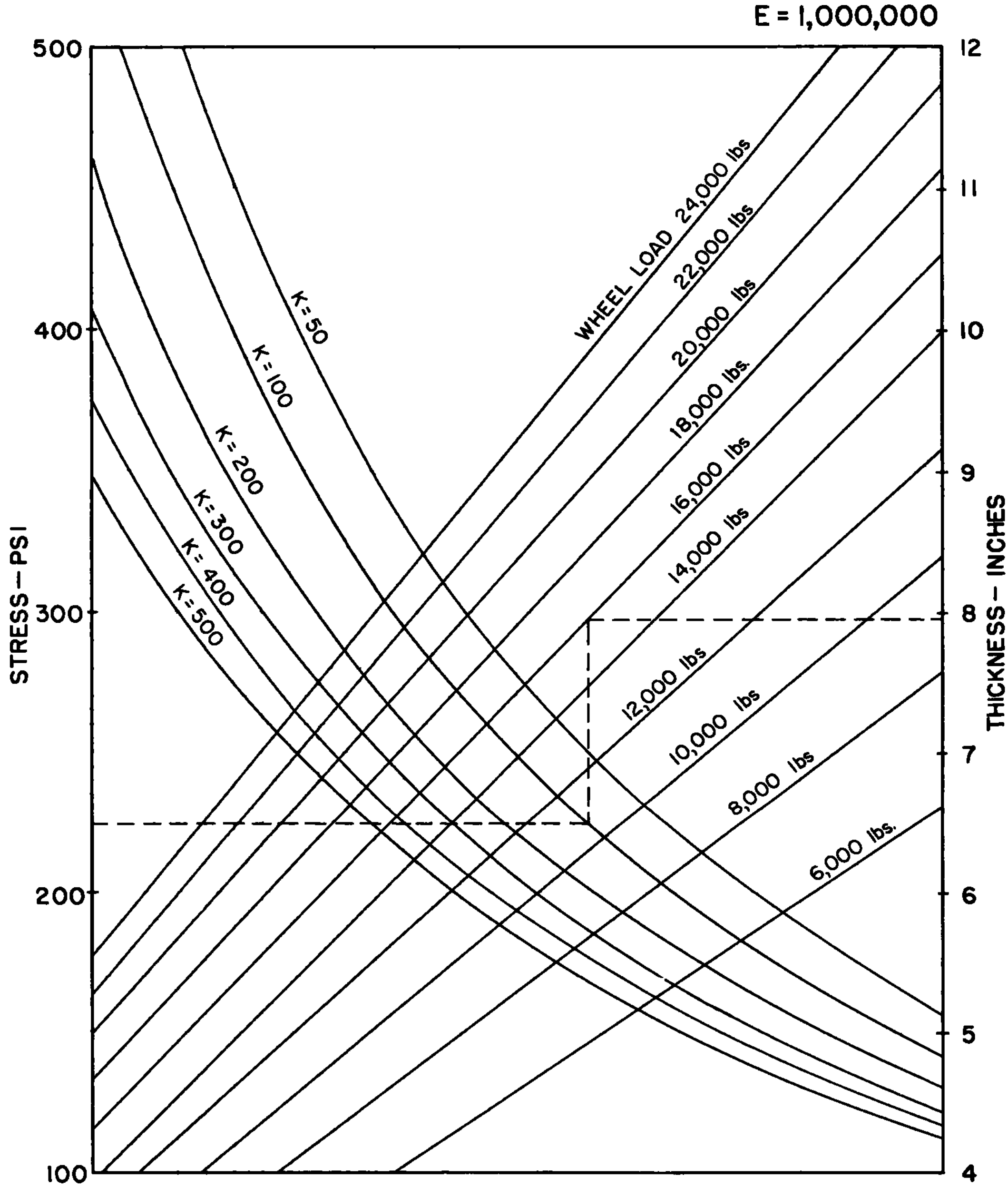
WESTERGAARD'S EDGE LOADING



DESIGN CHART FOR CONCRETE PAVEMENT
WESTERGAARD'S INTERIOR LOADING
(THEORETICAL)



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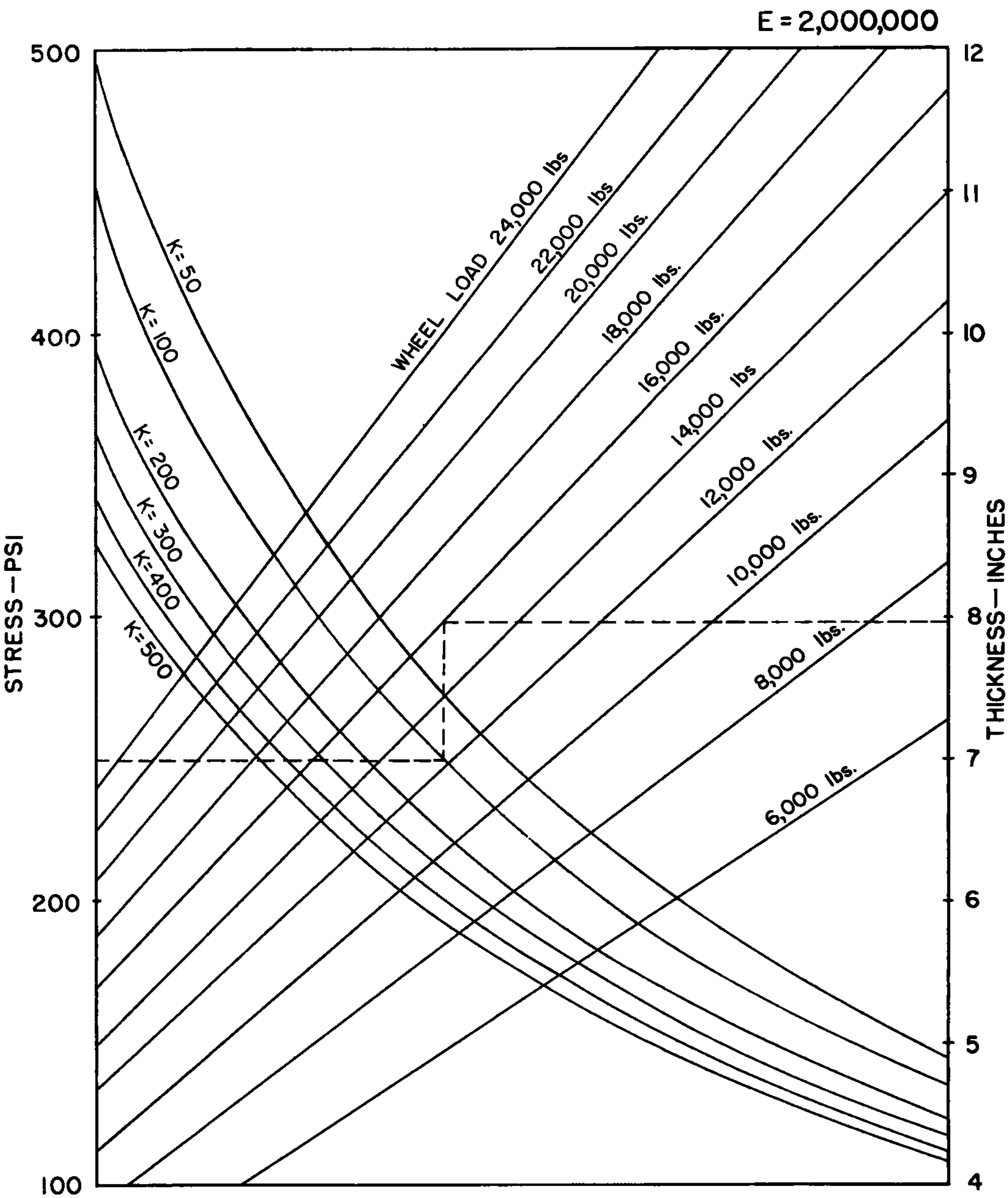

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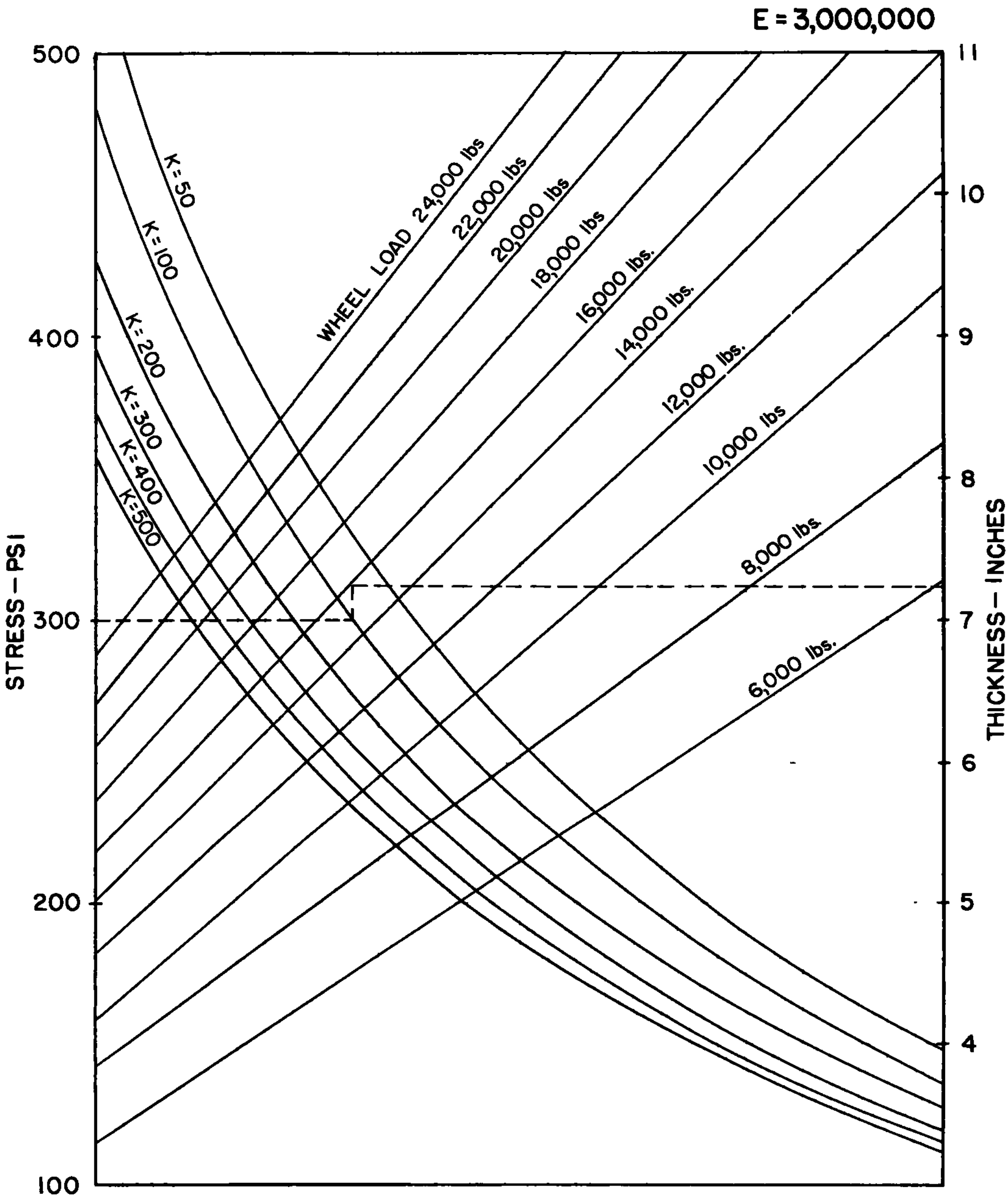

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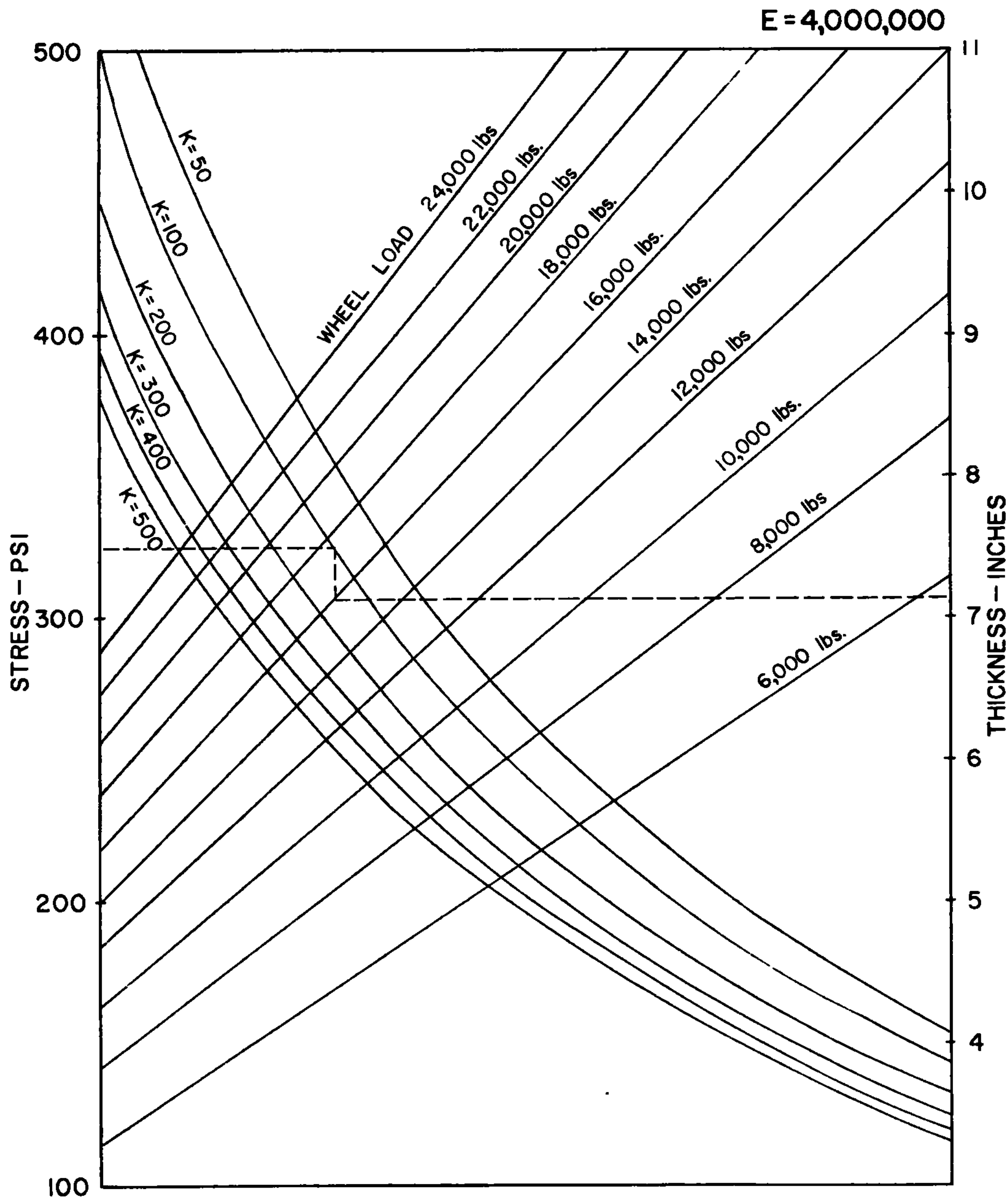

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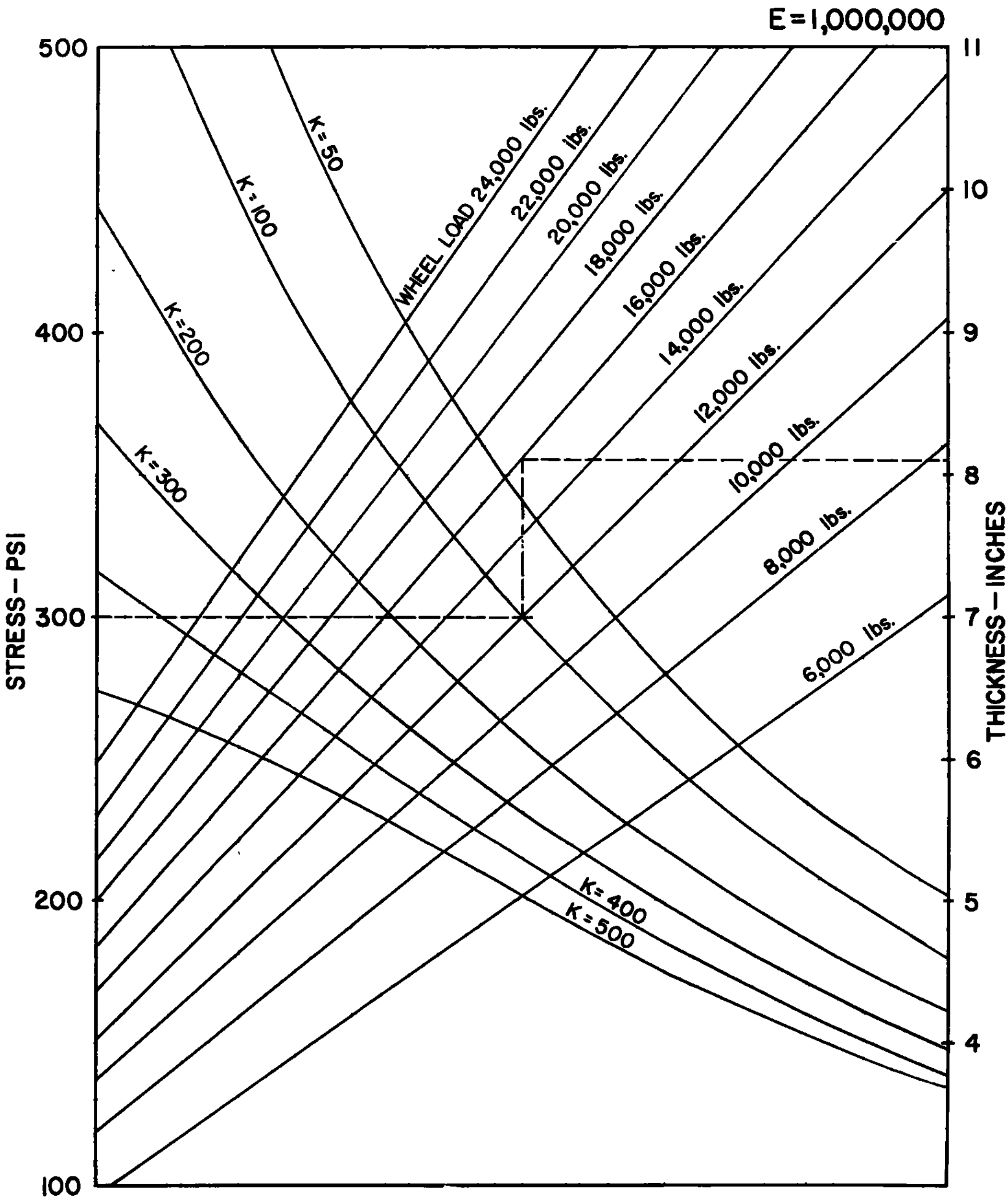
WESTERGAARD'S EDGE LOADING



An Experimental Continuously-Reinforced Concrete Pavement in Michigan

GENE R. CUDNEY, Physical Research Engineer, Michigan State Highway Department

This paper summarizes the construction, instrumentation, observations, and measurements associated with an experimental continuously-reinforced concrete pavement located on Interstate Highway 96, now designated US 16, near Portland, Michigan.

The experimental project includes two 24-ft roadways each containing two 12-ft lanes. Two types of reinforcing steel, deformed bar mat and welded wire mesh, each providing a steel ratio of approximately 0.6 percent, were used in the continuously-reinforced, 8-in. uniform pavement sections. The eastbound roadway is composed of a 2-mi section of continuous wire mesh, 0.7 mi of standard 9-in. uniform pavement with contraction joints spaced at 99-ft intervals, and a 2-mi section of continuous bar mat. The westbound roadway contains approximately 4 mi of continuously-reinforced pavement, 2 mi each of bar mat and wire mesh. Relief sections of 9-in. uniform pavement 493 ft long, consisting of eleven 1-in. expansion joints, were placed at the ends of the continuously-reinforced sections.

Construction methods and equipment are described, including construction joints in the continuously-reinforced sections. Various characteristics associated with the construction phase of the project are discussed including subgrade soil classification, concrete and air temperatures, concrete strength, and a record of construction progress.

Studies involved in comparing the performance of various project sections include longitudinal displacements of the ends, end regions and center of the continuously-reinforced sections; relative displacements of joints and selected cracks; crack patterns and formation; surface roughness; effects of traffic; performance of relief sections; and load-deflection behavior. In addition, one section each of bar met, wire mesh, and standard mesh reinforcement was instrumented with strain gages for determination of steel stress variation.

●THIS REPORT outlines the location and description, scope of study, general construction aspects, and the methods of instrumentation and measurements of an experimental continuously-reinforced concrete pavement. No attempt is made here to present performance data recorded so far or to present the results of such data.

The discussion consists of three parts: the first includes the description, location, and scope of the project. The second presents the general construction features. The third describes the methods of instrumentation and measurements, and the course of study to be carried out in the future. A selective bibliography lists literature on the subject of continuously-reinforced concrete pavement.

Appendices include: "Recommendations on Minimum Requirements for Tests and Observations" prepared by the Highway Research Board Subcommittee on Continuously-Reinforced Concrete Pavements; instrumentation and installation details for the instrumented steel reinforcement; and construction data and materials characteristics, including steel reinforcement properties, air temperature variation, concrete strength, construction progress data, and subgrade soil classification.

The organization of the program of observations and tests on this project coincides in general with the HRB "Recommendations" (App. A). Two other subjects included as additional features of the research project are the strain gage instrumentation of the steel reinforcement for strain analysis, and the load-deflection study of both the continuous and standard pavement sections.

LOCATION AND DESCRIPTION

After study and inspection of certain existing continuously-reinforced pavements, the Michigan State Highway Department authorized construction of an experimental pavement of this type in 1957, with the primary purpose of studying durability, construction efficienty, and costs, as compared to current standard pavement construction practice. The detailed design and layout of the pavement were made by the Bridge and Road Design Division. Instrumentation of the project during construction, and the necessary research and evaluation studies after construction, became the responsibility of the Research Laboratory Division.

With the approval of the Bureau of Public Roads, the experimental pavement was incorporated into the plans and specifications of Construction Project 34044, C7RN, located on the new Interstate Highway 96 (US 16 Relocation) between M 66 and Portland Road in Ionia County (Fig. 1).

The experimental pavement consists of both the eastbound and westbound roadways, each containing two 12-ft lanes, between Stations 838+00 and 1106+83.3 on the eastbound roadway, and Stations 861+96 and 1080+43.3 on the westbound. Both roadways are symmetrical with respect to grade and alignment, with a maximum grade of 0.95 percent. The pavement is straight except for a 1-deg, 30-min curve, 1,616 ft long, between Stations 1071+40 and 1087+56, and a similar curve beginning at Station 1100+40.03 in which the final 642 ft of the eastbound roadway is located. All continuously-reinforced

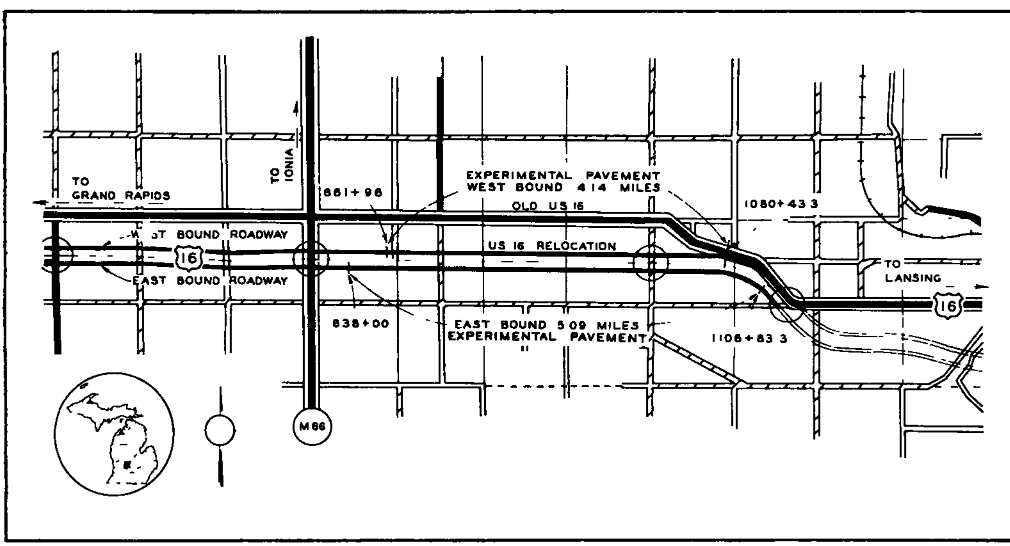


Figure 1. Location of experimental pavement.

sections are 8-in. uniform, and all standard reinforced sections are 9-in. uniform thickness.

The concrete mix was designed by the mortar voids method of proportioning, with a constant cement content of 5½ sacks per cu yd. Air entrainment of the concrete was provided by the addition of Darex AEA to the mix. The concrete had an average air content of 5.4 percent and an average slump of 2 in.

The entire pavement was placed on a 12-in. granular subbase overlying in general a Type A-4 clay subgrade. A typical road cross-section is shown in Figure 2.

The experimental pavement is composed of four distinct parts: continuously-reinforced sections with deformed bar mat, continuously-reinforced sections with welded wire mesh, a standard section with contraction joints spaced at 99 ft, and the relief sections at the ends of the continuously-reinforced portions. Drawings of the reinforcement and of a typical relief section are shown in Figures 3 and 4. The general plan and instrumentation layout of the entire test project is shown in Figure 5.

Eastbound Roadway

1. 10,550 ft of 8-in. uniform pavement continuously reinforced with wire mesh.
2. 3,804 ft of 9-in. uniform standard reinforced pavement with contraction joints at 99-ft intervals.
3. 10,550 ft of 8-in. uniform pavement continuously reinforced with bar mat.
4. Four relief sections each 493 ft long, of 9-in. uniform standard reinforced pavement.

Westbound Roadway

1. 10,331 ft of 8-in. uniform pavement continuously reinforced with wire mesh.
2. 10,530 ft of 8-in. uniform pavement continuously reinforced with bar mat.
3. Two relief sections each 493 ft long, of 9-in. uniform standard reinforced pavement.

Reinforcing Steel in Continuous Sections

Bar Mat. —Each half-mat is 6 ft 2 in. wide and 16 ft long, consisting of 11 No. 5 deformed bars in the longitudinal direction, and 7 No. 3 bars in the transverse direction, giving a steel percentage of 0.586.

Welded Wire Mesh. —Each wire mesh section is 11 ft 6 in. wide and 12 ft long, con-

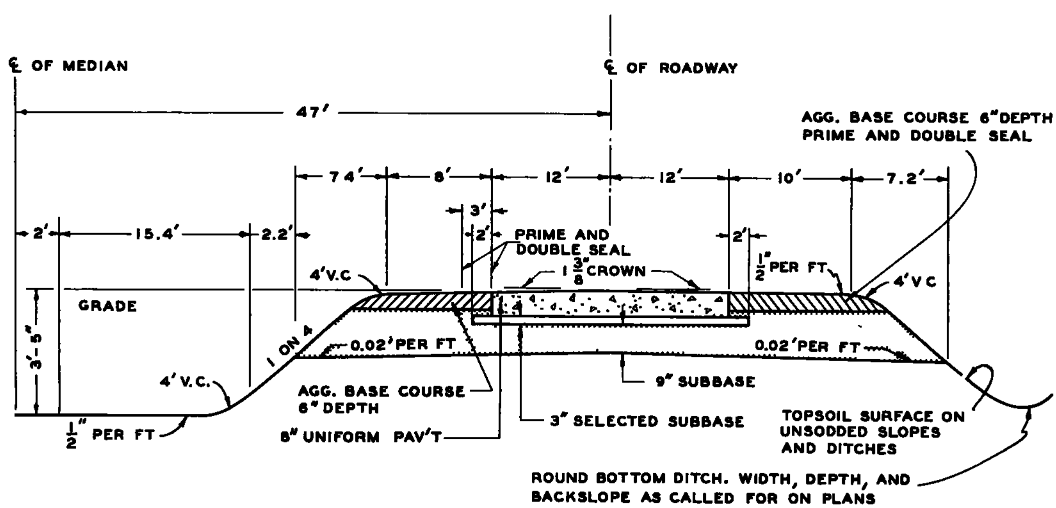


Figure 2. Typical road cross-section.

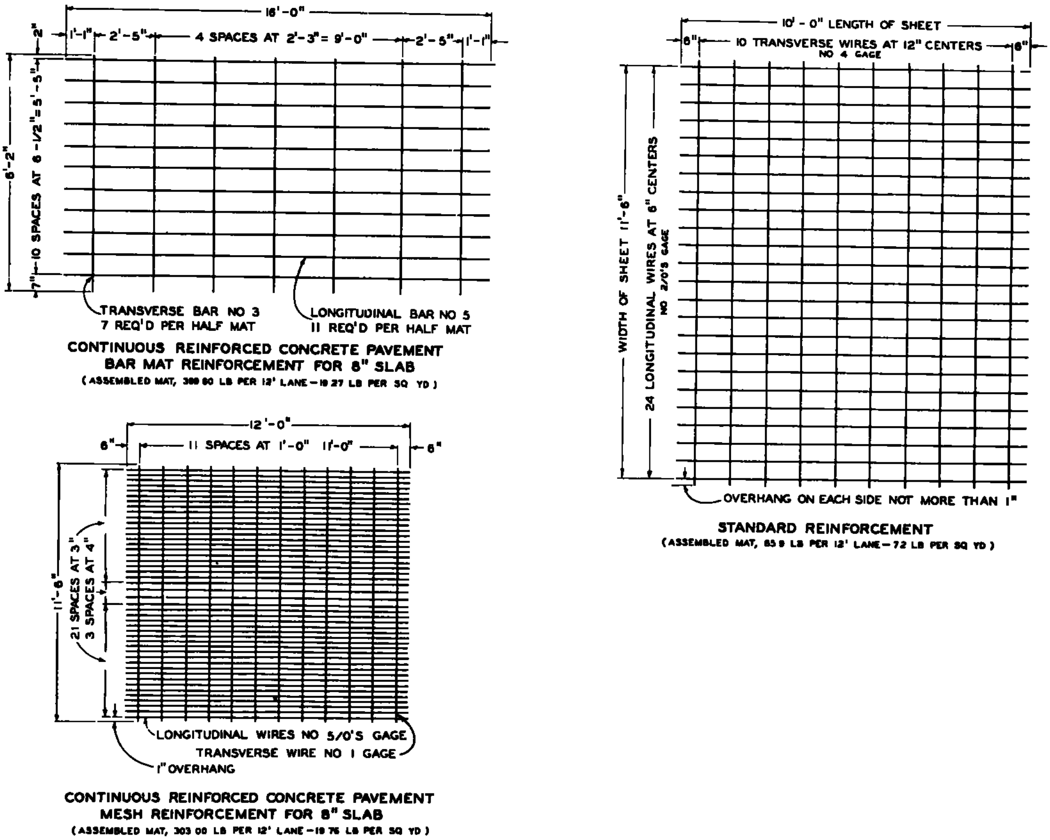


Figure 3. Detailed drawings of heavy mesh, bar mat, and standard reinforcement.

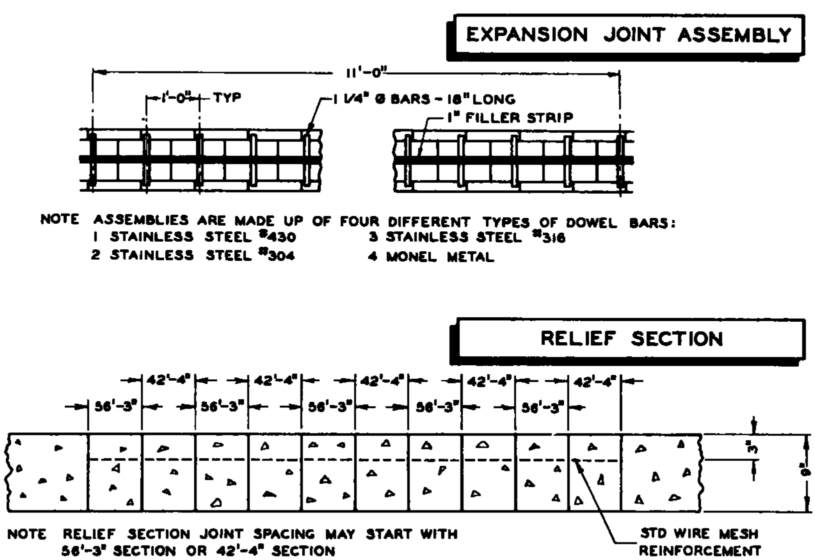


Figure 4. Typical expansion joint assembly and relief section.

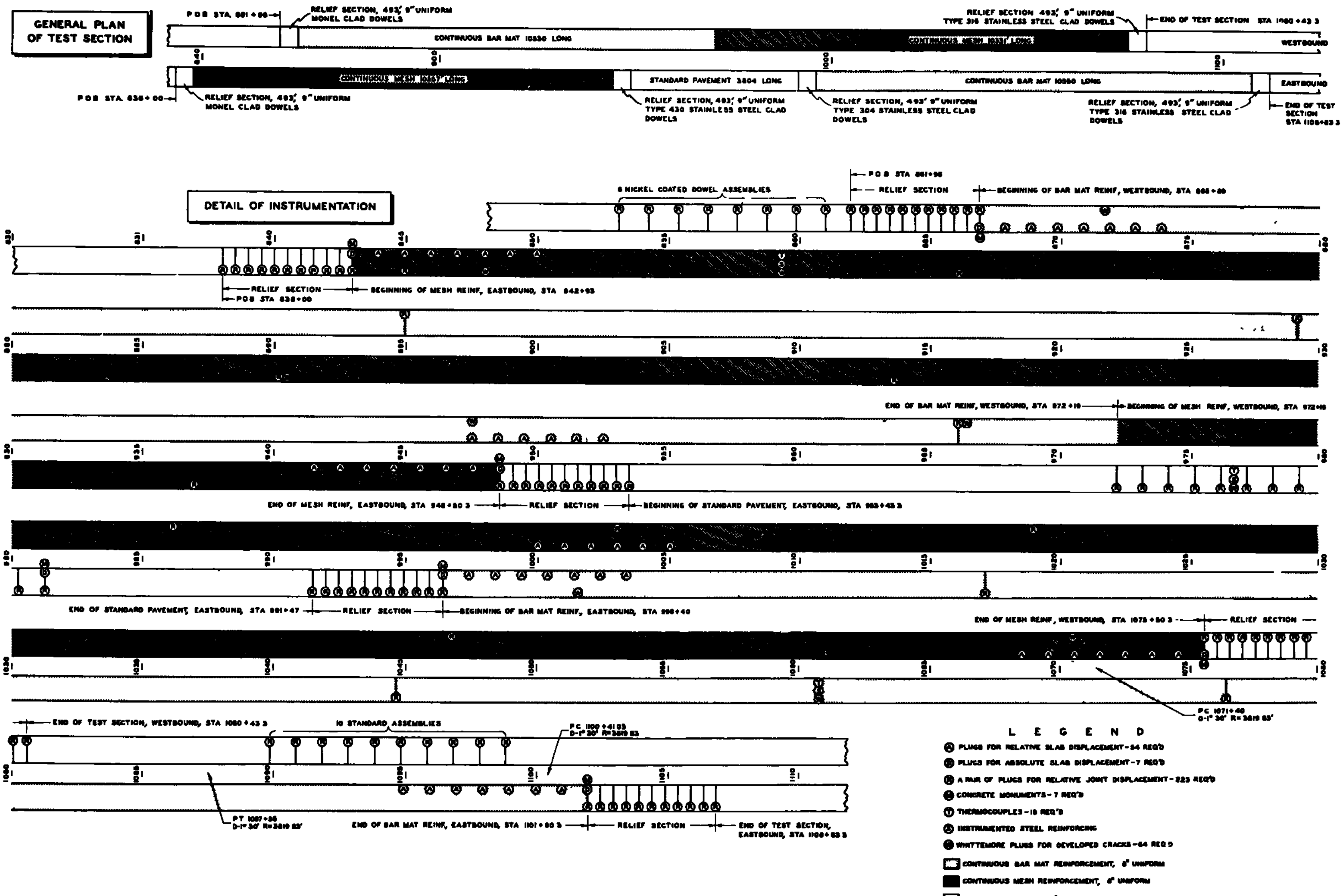


Figure 5. General plan and instrumentation layout.

sisting of 47 No. 5/0's gage wires in the longitudinal direction, and 12 No. 1 gage wires transversely, giving a steel percentage of 0.595.

Relief Sections

The six relief sections are each 493 ft long, of 9-in. uniform standard reinforced pavement, with eleven 1-in. expansion joints spaced alternately at 56 ft 3 in. and 42 ft 4 in. Load transfer dowel bars, 1¼ in. in idameter by 18 in. long, spaced at 12 in. intervals, were clad with corrosion resistant alloy sleeves to prolong service life and to provide more freedom of movement for the expansion joints in the relief sections. Four of the six relief sections contain~~ed one of three types of stainless~~ steel-clad bars, Types 304, 316, or 430, and the remaining two relief sections contained monel-clad dowel bars. The minimum sleeve thickness for the Type 430 stainless steel-clad bars was 0.015 in., while the Types 304 and 316 stainless steel and the monel-clad bars had a minimum sleeve thickness of 0.010 in. All the bars were coated with a cutback asphalt and inserted in standard 1-in. expansion joint assemblies prior to installation in the pavement.

In addition, eight consecutive contraction joints in a section of 9-in. uniform standard pavement outside the limits of the continuously-reinforced test pavement were composed of standard contraction joint assemblies, containing 1¼- by 18-in. nickel coated hot-rolled steel bars. Performance of this section, along with that of the 1-in. expansion joints in the six relief sections, will be studied as part of the Department's research project on dowel bar corrosion.

Scope of Study

In order to properly evaluate and compare the performance of the various sections of the project, the following factors are being studied:

1. Magnitude and variation of absolute longitudinal displacement of the ends of the continuously-reinforced sections.
2. Magnitude and variation of longitudinal displacements of the center and end regions of the continuously-reinforced sections.
3. Magnitude and variation of the relative longitudinal displacements of joints and cracks in both the standard and continuous sections.
4. Magnitude and variation of crack openings in various regions of the continuous sections.
5. Magnitude and variation of stresses in the bar mat, wire mesh, and standard reinforcement.
6. Initial surface roughness and roughness changes with time and traffic,
7. Static and dynamic load-deflection characteristics at various points in the standard and continuous sections.
8. Function of relief sections in relation to slab performance.
9. Effect of traffic on pavement performance in all sections.

To correlate these items with slab performance, the following studies are included:

1. Soil types and characteristics of the subbase and subgrade.
2. Physical properties of the reinforcing steel.
3. Physical properties of the concrete.
4. Steel stresses in relation to slab temperature variation.

Throughout the entire project, various measuring devices and equipment are being used:

1. SR-4 electrical resistance wire strain gages and an SR-4 static strain indicator for determining steel reinforcement strains.
2. Thermocouples for slab temperature determination.
3. Reference monuments and a displacement device for determining the movements of the ends of continuous pavement sections.

4. Reference plugs for measurement of relative joint and crack displacement.
5. Vernier calipers and an invar tape for measurement of relative joint displacement and sectional slab displacement.
6. Whittemore mechanical strain gage and a scale microscope for measurement of crack openings.

PAVEMENT CONSTRUCTION

Construction of the experimental pavement began September 22, 1958, and paving operations were completed October 20, 1958. Double lane construction was employed, whereby the entire 24-ft width of pavement was placed at one time. No. 4 deformed bars, 30 in. long, were spaced at 40 in. on all sections of the test pavement as transverse tie bars between the two 12-ft lanes, with the longitudinal centerline joint sawed and sealed at a later time. All the contraction joints in the standard section contained load transfer assemblies consisting of 1¼-in. diameter bars 18 in. long, spaced at 12 in. centers.

The steel reinforcement was placed 3 in. below the surface in the 8-in. uniform continuous sections, as well as in the 9-in. uniform standard section. The steel in the continuous bar mat sections was lapped 13 in., with the ends of the longitudinal bars placed against the last transverse bar of the preceding mat. The continuous mesh was lapped 12 in., so that the first transverse wire of the mesh being laid rested behind the last transverse wire of the preceding mesh section. The laps in both the bar mat and wire mesh reinforcement are shown in Figures 6 and 7.

The continuous mesh reinforcement was transported to the construction area on trucks or flatbed trailers, and placed in the pavement from the trucks as required. The bar mat reinforcement was spread out in piles approximately 75 ft apart along the shoulder slopes in advance of construction, where it would be readily available as needed.

In constructing the pavement slab, the contractor used two Koehring 34-E dual drum mixers, a Blaw-Knox spreader, two Jaeger-Lakewood finishing machines, and a Heltzel Flexplane. The maximum pavement lengths attained in a day's pour were 3,100 ft in the continuous mesh sections and 3,500 ft in the bar mat sections. The average lengths poured in a day were 2,500 ft for the continuous mesh and 3,086 ft in the bar mat.

Sequence of Operations

The sequence of construction operations for both the standard and continuously-reinforced sections (Figs. 8-17) was as follows:

1. Placing concrete on subbase from first mixer.

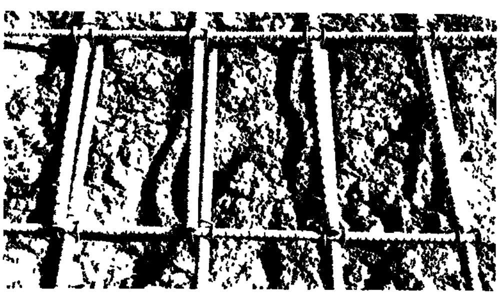


Figure 6. Lap in continuous bar mat reinforcement.

Figure 7. Lap in continuous wire mesh reinforcement.

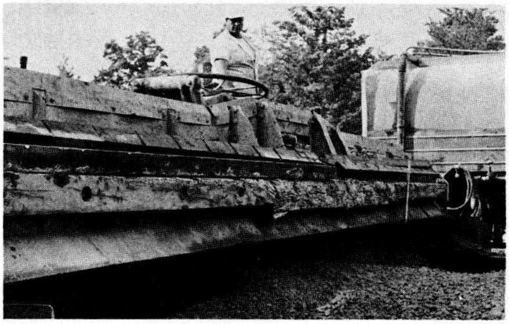


Figure 8. Spreading and striking off concrete with Blaw-Knox spreader.

Figure 9. Placing wire mesh reinforcement in continuous section.

Figure 10. Placing bar mat reinforcement in continuous section.

Figure 11. Placing final layer of concrete on bar mat reinforcement.

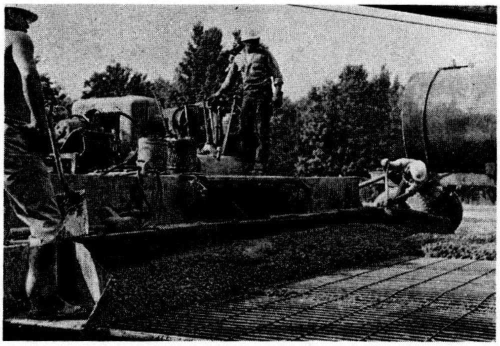


Figure 12. Spreading concrete with Jaeger-Lakewood finishing machine in continuous wire mesh section.

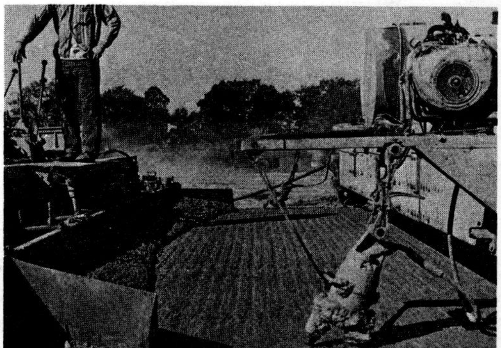


Figure 13. Initial finishing of concrete surface with Jaeger-Lakewood finishing machine.

2. Spreading and striking off concrete 3 in. below finished surface with Blaw-Knox spreader.
3. Placing steel reinforcement.
4. Placing transverse tie bars.
5. Placing final layer of concrete from second mixer.

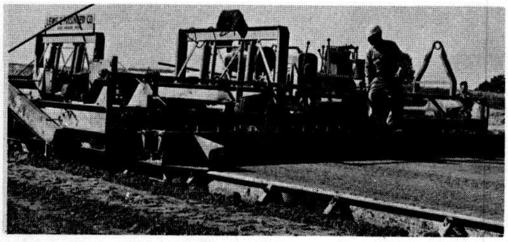


Figure 14. Final machine finishing with Heltzel Flexplane.

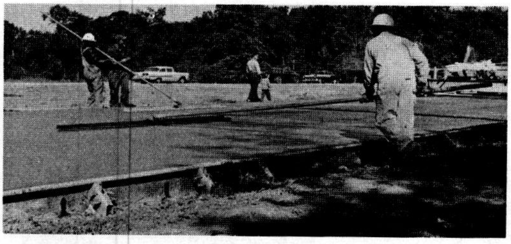


Figure 15. Final hand finishing of concrete surface.

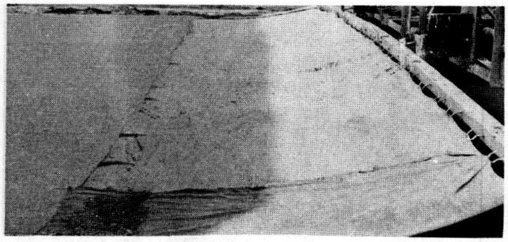


Figure 16. Applying burlap drag finish to concrete surface.

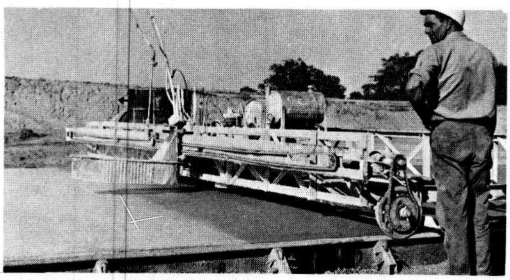


Figure 17. Applying white membrane curing compound.

6. Spreading and screeding concrete with Jaeger-Lakewood finishing machine.
7. Initial finishing of concrete surface with second Jaeger-Lakewood machine.
8. Final machine finishing of concrete surface with Heltzel Flexplane.
9. Final hand finishing of concrete surface.
10. Applying burlap drag finish to concrete surface.
11. Applying white membrane curing compound.

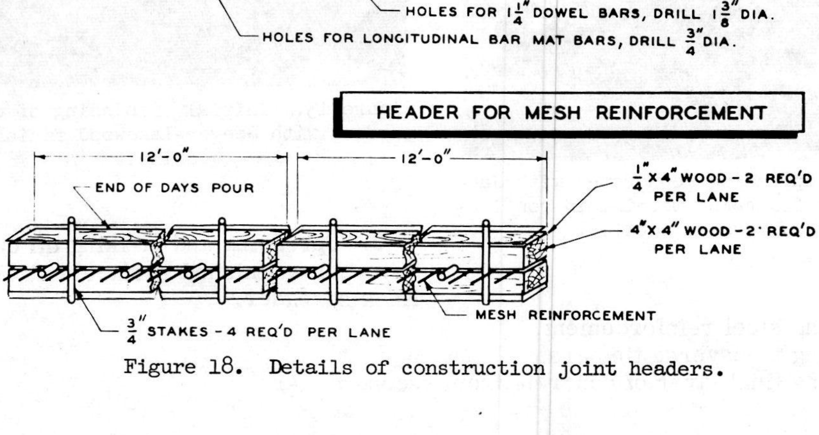


Figure 18. Details of construction joint headers.

Construction Joints

Each construction joint in the continuously-reinforced sections consisted of a header formed with two sets of 12-ft long 4 by 4's with a ¼- by 3⅝-in. wooden strip nailed to each piece. The reinforcement was carried through the joint for a minimum distance of 5 ft, with the wire mesh sandwiched between the two sets of 4 by 4's, and the bar mat extended through holes drilled at the center of the 4 by 4's to accommodate the individual bars. In addition to the pavement reinforcement through the joint, 1¼-in. diameter by 18-in. steel dowel bars spaced at 12 in. were placed in holes drilled through the centers of the adjoining 4 by 4's. The dowels were wired to the reinforcement to maintain their proper position. The joint header is shown in Figure 18.

The portion of the reinforcement extending through the joint was supported on boards which kept the steel level and made it easier to align the dowel bars. The concrete was hand vibrated throughout the width of the joint and about 5 ft back along the side forms. Burlap was placed over the extended reinforcement to catch excess concrete spilling over the header during finishing operations. Construction is shown in Figures 19 through 24.

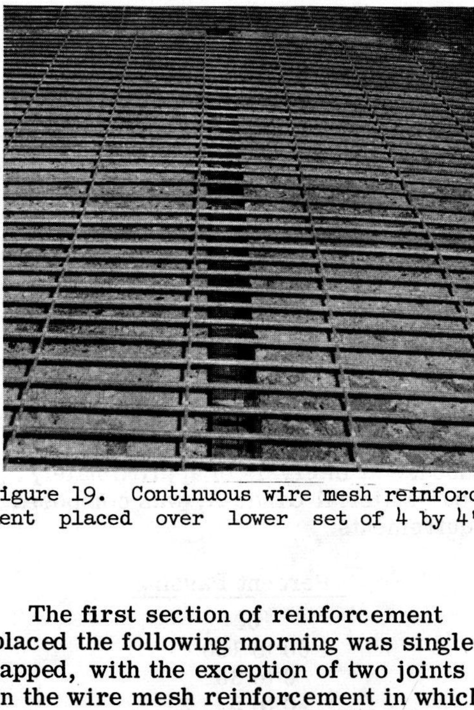


Figure 19. Continuous wire mesh reinforcement placed over lower set of 4 by 4's.

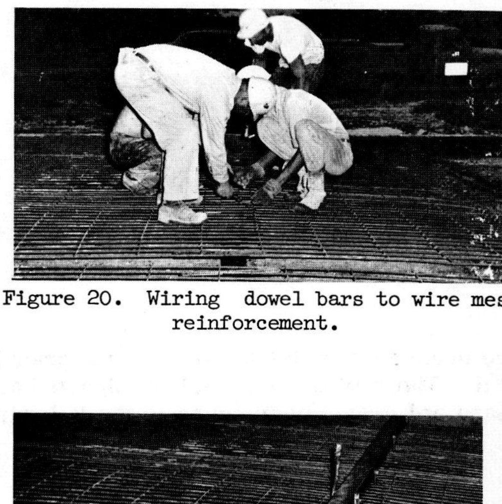


Figure 20. Wiring dowel bars to wire mesh reinforcement.

The first section of reinforcement placed the following morning was single lapped, with the exception of two joints in the wire mesh reinforcement in which the steel was double lapped. In addition, two joints in the bar mat and two in the wire mesh sections contained dowel bars previously coated with an RC-1 cutback asphalt. The remaining joints contained uncoated bars.

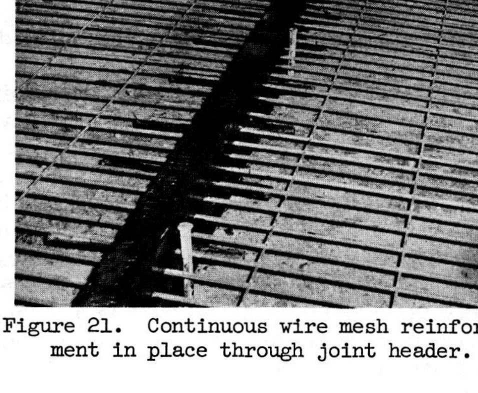


Figure 21. Continuous wire mesh reinforcement in place through joint header.

INSTRUMENTATION AND MEASUREMENTS

This discussion pertains to the methods of instrumentation and observations carried out to determine and evaluate the various factors involved in the study.

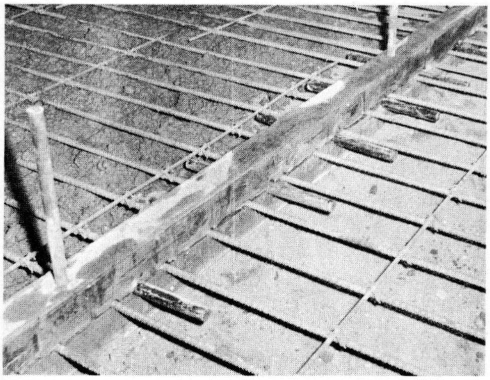


Figure 22. Continuous bar mat reinforcement in place through joint header.

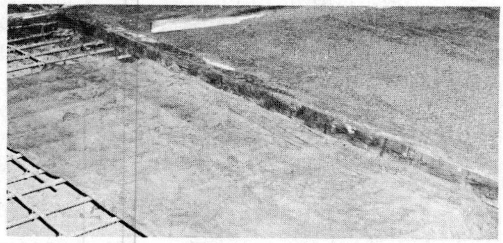


Figure 23. Burlap covering on extended bar mat reinforcement.

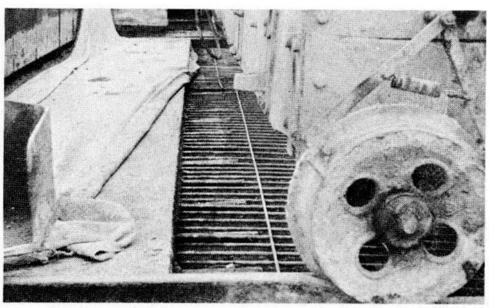


Figure 24. Typical joint in wire mesh reinforcement after concrete set.

It is divided into four phases, encompassing preparations prior to construction, observations and instrumentation during construction, measurements and instrumentation after construction, and finally, the course of study to be pursued in the future. The description of each phase includes the measurements, observations, and instrumentation involved.

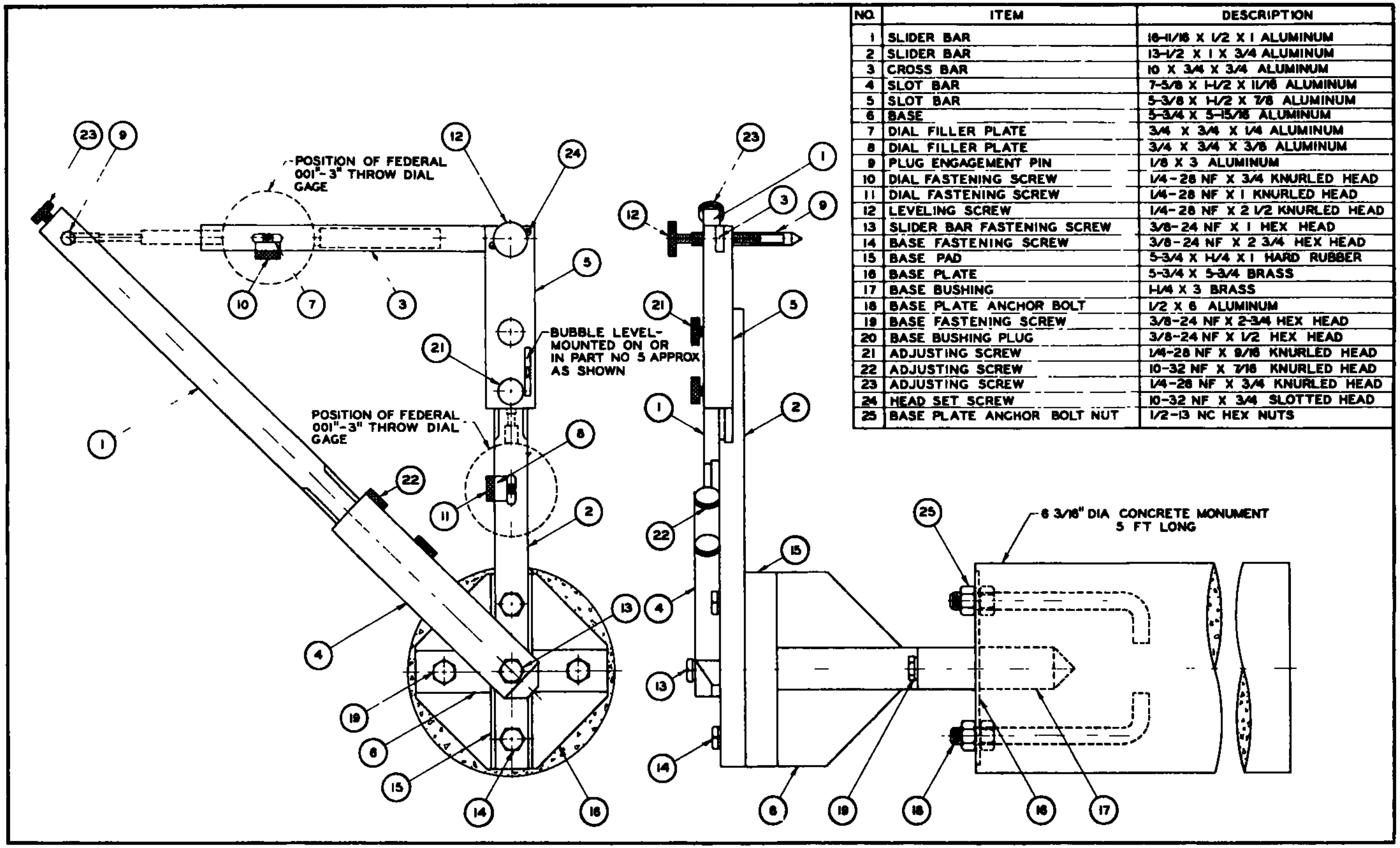
Preparations for Construction

Soil Characteristics.—To correlate soil types and characteristics with pavement performance, soil samples were obtained from the finished subgrade at intervals not exceeding 1,000 ft, and where soil changed type throughout both roadways. A subgrade soil classification chart based on the AASHO classification system is shown in Figure 49 (App. C). In addition, density tests were made on samples taken from the granular subbase at intervals of approximately 400 ft. The subbase material, designated as Porous Material Grade A, was composed of sand and gravel conforming to the following requirements:

Sieve Size	Percent Passing
1 in.	100
2½ in.	60 to 100
No. 100	0 to 30
Loss by washing	0 to 5

The average dry density of the subbase, as determined by the Michigan Cone Test, was 118 pcf.

Absolute Slab Displacement.—To determine the absolute displacement of the ends of the continuous pavement sections, a device was constructed consisting essentially of three movable arms forming a right triangle, with two 0.001-in. Federal dials fixed to two of the arms (Fig. 25). This instrument, when attached to a fixed base and leveled, provides a means of measuring both the longitudinal and lateral displacement of a point on the concrete surface. In conjunction with this instrument, a steel reference bar was made to determine temperature deformations of the device itself, and also to check on the initial settings of the dial gages. Seven reinforced concrete monuments, $6\frac{3}{16}$ in. in diameter and 5 ft long, were constructed to provide fixed points for measurements of absolute slab movement. The top of each monument contains a brass plate, with brass bushings to accommodate the base of the pavement displacеometer.



NO.	ITEM	DESCRIPTION
1	SLIDER BAR	18-11/16 X 1/2 X 1 ALUMINUM
2	SLIDER BAR	13-1/2 X 1 X 3/4 ALUMINUM
3	CROSS BAR	10 X 3/4 X 3/4 ALUMINUM
4	SLOT BAR	7-5/8 X 1-1/2 X 11/16 ALUMINUM
5	SLOT BAR	5-3/8 X 1-1/2 X 7/8 ALUMINUM
6	BASE	5-3/4 X 5-15/16 ALUMINUM
7	DIAL FILLER PLATE	3/4 X 3/4 X 1/4 ALUMINUM
8	DIAL FILLER PLATE	3/4 X 3/4 X 3/8 ALUMINUM
9	PLUG ENGAGEMENT PIN	1/8 X 3 ALUMINUM
10	DIAL FASTENING SCREW	1/4-28 NF X 3/4 KNURLED HEAD
11	DIAL FASTENING SCREW	1/4-28 NF X 1 KNURLED HEAD
12	LEVELING SCREW	1/4-28 NF X 2 1/2 KNURLED HEAD
13	SLIDER BAR FASTENING SCREW	3/8-24 NF X 1 HEX HEAD
14	BASE FASTENING SCREW	3/8-24 NF X 2 3/4 HEX HEAD
15	BASE PAD	5-3/4 X 1-1/4 X 1 HARD RUBBER
16	BASE PLATE	5-3/4 X 5-3/4 BRASS
17	BASE BUSHING	1-1/4 X 3 BRASS
18	BASE PLATE ANCHOR BOLT	1/2 X 6 ALUMINUM
19	BASE FASTENING SCREW	3/8-24 NF X 2-3/4 HEX HEAD
20	BASE BUSHING PLUG	3/8-24 NF X 1/2 HEX HEAD
21	ADJUSTING SCREW	1/4-28 NF X 9/16 KNURLED HEAD
22	ADJUSTING SCREW	10-32 NF X 7/16 KNURLED HEAD
23	ADJUSTING SCREW	1/4-28 NF X 3/4 KNURLED HEAD
24	HEAD SET SCREW	10-32 NF X 3/4 SLOTTED HEAD
25	BASE PLATE ANCHOR BOLT NUT	1/2-13 NC HEX NUTS

Figure 25. Assembly drawing of pavement displaceometer.

Gage Plugs. —All the gage plugs used in conjunction with measurement of joint and slab displacements, and crack opening variations, were formed from ¼-in. diameter stainless steel countersunk head rivets, with appropriately machined conical holes or scratched cross hairs in the rivet heads.

Physical Properties of the Reinforcing Steel. —Five specimens each of the continuous wire mesh, bar mat, and standard mesh reinforcements were sampled and subjected to tensile tests to determine yield stress, ultimate stress, and percentage elongation characteristics. Physical properties of the three types of steel reinforcement are shown in Figure 50 (App. C).

Instrumented Reinforcing Steel. —Type A-12 SR-4 strain gages were placed on the three types of steel reinforcement as follows:

1. Bar Mat. Gages placed on five longitudinal bars, so that when a complete mat was in place in the 12-ft traffic lane, the fourth and seventh bars from each lane edge, and the eleventh bar from the outside lane edge were instrumented.
2. Continuous Wire Mesh. Gages placed on five longitudinal wires, so that when a complete mesh section was in place in the 12-ft traffic lane, the fifth and fourteenth wires from each lane edge, and the twenty-third wire from the outside lane edge were instrumented.
3. Standard Wire Mesh. Gages placed on five longitudinal wires in same positions as for bar mat.

A detailed description of the instrumentation and installation for the instrumented steel reinforcement may be found in Appendix B, and placement of the strain gages is shown in Figure 26.

Induced Cracks at Instrumented Steel Locations. —To insure formation of a crack at each of the instrumented steel locations, three units were constructed, each formed of two 12-ft lengths of No. 28 gage corrugated steel, 3 in. high, welded to a piece of No. 20 gage sheet steel. This corrugated crack former is shown in Figure 26.

Operations During Construction

The operations described here are shown in Figures 27 through 40.

Air Temperature. —A record of daily air temperature throughout the construction period was obtained by means of a Honeywell automatic temperature recorder. Table 1 (App. C) gives the high, mean, and low temperatures for each 24-hr day throughout the construction period, plus an additional 11 days after the experimental pavement was completed.

Concrete Temperature and Steel Depth. —The mid-depth concrete temperature and steel depth were measured at two points in each lane at intervals of 200 ft throughout the entire test pavement. These measurements were taken just before the final hand finishing operation.

Concrete Properties. —One test cylinder and one beam for modulus of rupture tests were taken at intervals of approximately 600 ft throughout the test pavement. One-half these samples were tested at 7 days and one-half at 28 days. Other pertinent concrete characteristics were determined and recorded in the daily construction reports. Modulus of rupture and compressive strength values, as well as daily construction progress and air temperature during construction, are given in Table 2 (App. C).

Construction Conditions. —All events pertinent to construction and the possible effects on pavement performance were recorded each day throughout the construction period.

Placing Plugs for Relative Joint Movement. —A set of gage plugs was placed at each joint in the six relief sections, at all construction joints in the continuously-reinforced sections, and at ten consecutive contraction joints in the standard pavement section. All plugs were placed in the concrete just after the final hand finishing operation, 4 in. each side of the joint centerline and 12 in. from the pavement edge in the traffic lane.

Placing Plugs for Absolute Slab Movement. —One gage plug was placed in the concrete after the final hand finishing operation, 4 in. from the ends of the continuously-reinforced sections and 12 in. from the pavement edge in the passing lane. Similarly, a gage plug was installed 4 in. from the beginning joint of the ten consecutive contraction joints in the standard section.

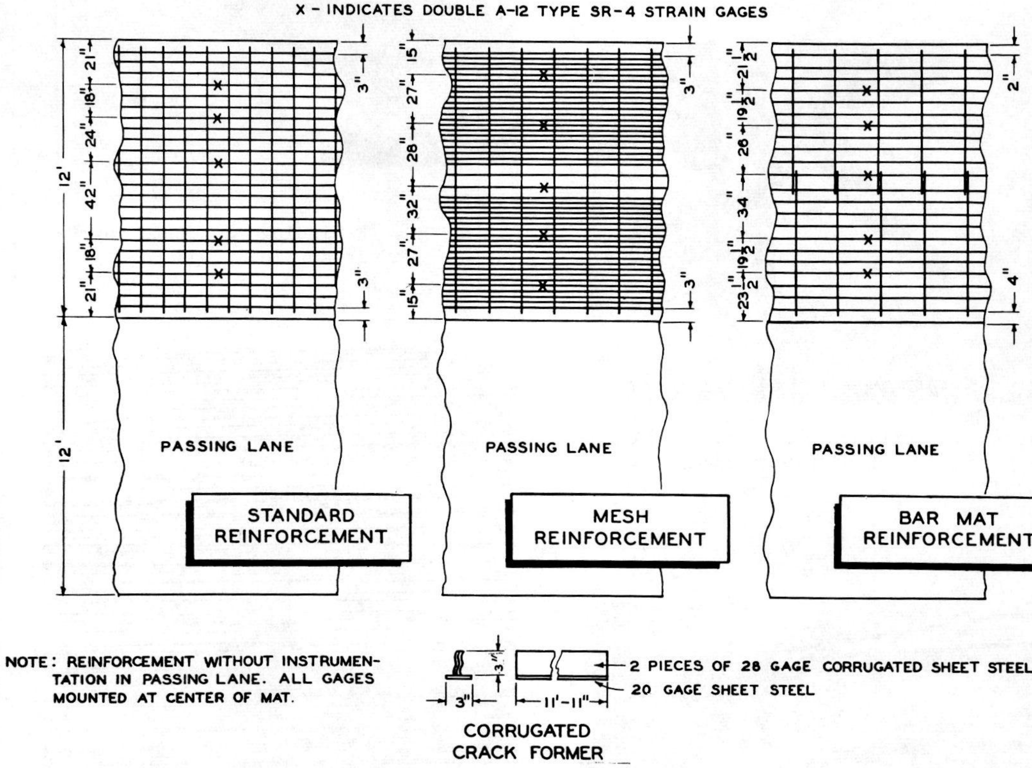


Figure 26. Details of strain gage placement and corrugated crack former.

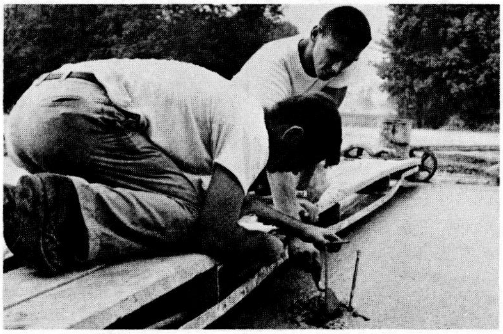


Figure 27. Taking slab temperature and steel depth measurements.

Figure 28. Taking beam samples for modulus of rupture tests.

Placing Plugs for Sectional Slab Displacement. —A set of seven plugs, beginning at the ends of the continuously-reinforced sections and spaced 99 ft apart for 693 ft, were placed 12 in. from the pavement edge in the passing lane. In addition, a set of five plugs, similarly spaced, were placed in the center 495-ft region of a day's pour in each of the bar mat and wire mesh continuous sections.

Placing Instrumented Steel. —The instrumented continuous reinforcement was placed in the traffic lane, approximately in the center region of a day's pour. The instrumented standard mesh was placed in the traffic lane halfway between contraction joints. In

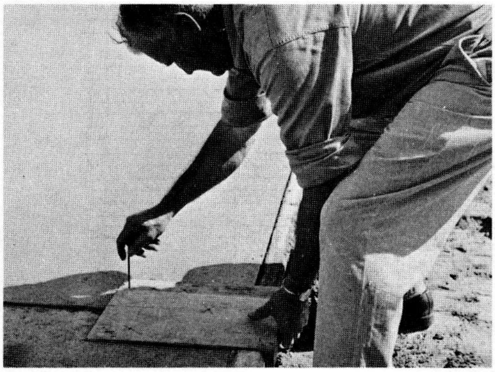


Figure 29. Setting plugs for differential slab displacements.

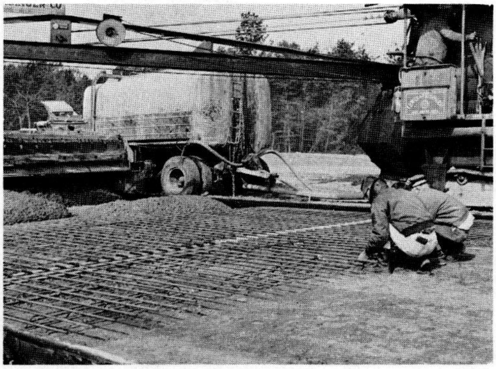


Figure 31. Instrumented bar mat reinforcement in place.

Figure 33. Placing plastic tube containing thermocouples in subbase.

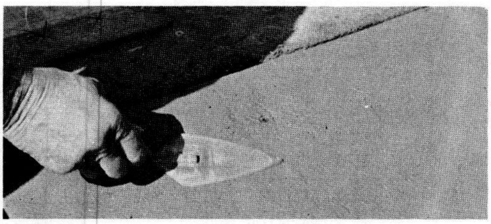


Figure 30. Finishing concrete surface after setting plugs.

Figure 32. Instrumented wire mesh reinforcement in place.

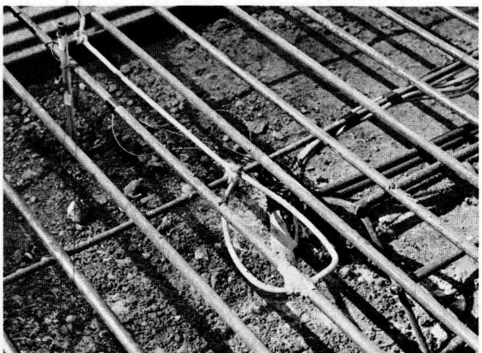


Figure 34. Instrumented wire mesh reinforcement with corrugated crack former and thermocouples.

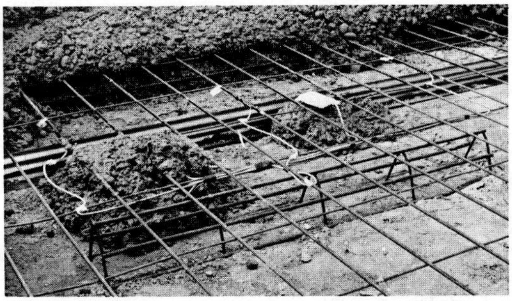


Figure 35. Instrumented standard mesh reinforcement with corrugated crack former.

Figure 36. Spreading concrete from bucket behind instrumented standard mesh reinforcement.

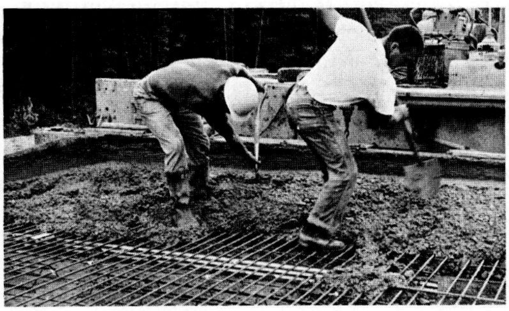


Figure 37. Placing concrete around corrugated crack former in heavy mesh reinforcement.

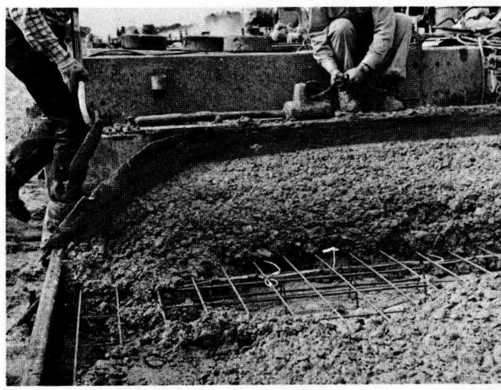


Figure 38. Pushing concrete into instrumented standard mesh reinforcement.

Figure 39. Vibrating concrete adjacent to instrumented standard mesh reinforcement.

all cases, the reinforcement (including the passing lane) was supported on wire chairs in advance of construction. Each wire or bar was taped for the same length as those with attached SR-4 strain gages, to maintain the same bonding characteristics across the 24-ft pavement. Concrete was poured and spread up to the instrumented section, which was then bypassed. Next, regular reinforcement was laid, working backward from the instrumented steel for approximately 50 ft, to insure a proper lap in the immediate vicinity of the instrumented reinforcement. The concrete was carefully placed over the instrumented reinforcement to prevent any gage damage. The concrete area adjacent to the transverse gage line of the reinforcement was then vibrated across the full pavement width.

Placing Thermocouples.—Two sets of three thermocouples each were placed at each

Figure 40. Wiring strain gage and thermocouple leads at junction box.

of the three instrumented steel locations previously described. Each set was placed 12 in. from the corrugated crack former and 18 in. from each edge of the traffic lane. The thermocouples were placed in a plastic tube so that one was 1 in. from the slab bottom, one at mid-depth, and one 1 in. from the top surface.

Operations After Construction

Operations during the months immediately following construction are shown in Figures 41 through 48.

Joint and Slab Measurements.—Initial readings were obtained on all expansion joints in the relief sections, all construction joints in the continuous sections, and the ten construction joints in the standard section just after initial set of the concrete, with a Starrett 0.001-in. vernier caliper.

The concrete monuments were set in the shoulder 5 in. from the pavement edge at the previously described seven locations, within two days after the concrete slab had been poured. Initial measurements were made at this time with the pavement displaceometer.

Incremental displacements of the six end regions and two central areas in the continuous sections were measured with a 100-ft invar tape coupled with the vernier calipers, the day after each section had been poured. In taking these measurements, a

Figure 41. Measuring relative joint displacement in relief section with vernier calipers.

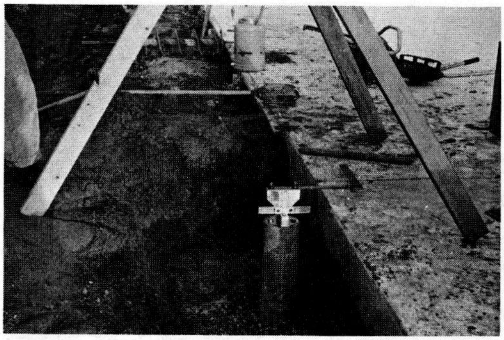


Figure 42. Setting monument for absolute slab displacement.

Figure 43. Measuring absolute slab displacement with pavement displaceometer.

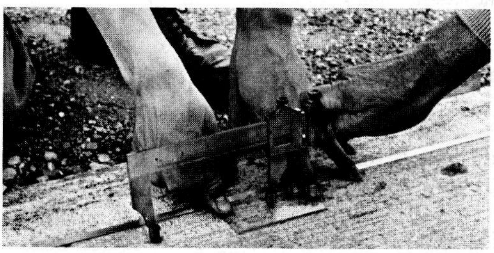


Figure 44. Measuring sectional slab displacement with vernier calipers and invar tape.

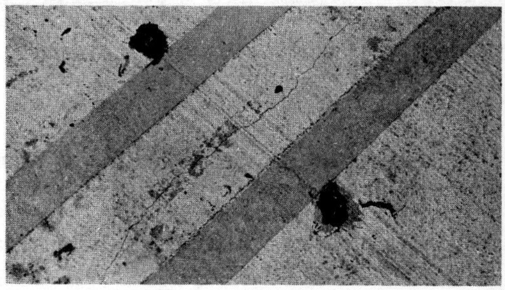


Figure 45. Plugs set in epoxy resin for crack opening measurements.

Figure 46. Measuring surface crack opening with graduated scale microscope.

Figure 47. Measuring crack opening with Whittemore strain gage.

thin plastic plate with a conical hole and etched cross hairs is held on the mark of the invar tape. One end of the caliper fits into the gage plug, and the other sits in the plastic plate to obtain the reading.

Crack Survey.—A continuous accumulative crack survey was conducted on the entire test pavement to determine the crack pattern, so that each day's pour was surveyed daily for the first five days and then on approximately the eighth, twelfth, and sixteenth days thereafter.

Crack Measurements.—Eight sets of four cracks each were selected in the following regions of continuous reinforcement:

1. Near a construction joint in each of the continuous mesh and bar mat sections.
2. Two sets each approximately 500 ft from the ends of the bar mat and continuous wire mesh sections.

Figure 48. MSHD roughometer truck.

3. One set in the center section of a day's pour in each of the bar mat and continuous mesh sections.

The first cracks appearing in these areas were selected and two gage plugs installed 5 in. each side of each crack, 12 in. from the pavement edge in the traffic lane. The plugs were set in Armstrong Type A-1 cement, and the initial reading taken about an hour later with a Whittemore 10-in. length mechanical strain gage. The average surface width of the crack across the gage line was also measured using a scale microscope graduated to 0.004 in. This reading was applied as a correction to the initial Whittemore measurement. In addition, plugs were set and measurements taken in the same manner at the instrumented steel locations as soon as the cracks appeared.

Strain and Temperature. —All strains and slab temperatures at the three instrumented steel sections were recorded daily until the induced cracks formed, then weekly until January 15, 1959. In addition, strain and temperature measurements were obtained for a 24-hr period after each induced crack formed.

Surface Roughness. —The initial surface roughness indices for the various sections of the test pavement were obtained April 1, 1959, approximately three months after the pavement was opened to traffic.

Crack Width Variation. —Cores were taken in July 1959, at two of the wider cracks in each of the bar mat, continuous mesh and standard mesh reinforced pavement areas.

Load-Deflection Tests. —Load-deflection studies involving static and dynamic truck loadings were made in separate duplicate day and night tests in September 1959. Locations included a construction joint and an adjacent crack in the bar mat and the mesh sections, two points halfway between two adjacent cracks and at a crack in both the mesh and mat central regions, a contraction joint and two points midway between two contraction joints in the standard pavement. Deflection of these 13 points was measured at the outside edge of the traffic lane in all cases.

Subsequent Course of Study

The various factors included in this project will be evaluated in continuing observations over a period of years or until sufficient data have been obtained to warrant conclusions.

Air Temperature. —The average monthly air temperature in the vicinity of the test site will be obtained throughout the project test period through a local station of the United States Weather Bureau.

Pavement Condition. —The test pavement will be inspected on the 15th of January, April, July, and October each year throughout the test period. Photographs will be taken to record pavement performance characteristics.

Surface Roughness. —Surface roughness will be determined once each year throughout the life of the project.

Crack Survey. —A crack survey will be made on the 15th of January, April, July, and October of each year throughout the test period.

Traffic Survey. —A traffic survey to determine axle weights and frequencies will be made once every three years throughout the test period.

Relative Joint Displacements. —The relative displacement of all expansion joints in relief sections, ten contraction joints in the standard section, and all construction joints in the continuous sections will be measured on the 15th of January, April, July, and October of each year throughout the test period.

Slab Displacements. —The absolute movement of the ends and relative slab displacements in the center and end regions of the continuous sections will be measured on the 15th of January, April, July, and October of each year throughout the test period.

Strain and Temperature Measurements. —Strains and slab temperatures of the three instrumented steel reinforced sections will be measured biweekly throughout the life of the strain gage and thermocouple instrumentation.

Crack Measurements. —Surface width of the selected cracks in the end and center regions, and near construction joints in the continuously-reinforced sections will be measured on the 15th of January, April, July, and October of each year throughout the test period. The cracks formed at the three instrumented steel reinforcement locations will be measured biweekly in conjunction with the regular strain and temperature readings.

Crack Width Variation. —Cores will be taken at two of the wider cracks in each of the bar mat, continuous mesh, and standard mesh reinforced areas at 5-yr intervals throughout the life of the project.

Pictorial Crack Record. —A progressive series of photographs of typical cracks in the center and end regions, and near construction joints in each of the bar mat and wire mesh continuous sections, will be taken each fall throughout the life of the project.

ACKNOWLEDGMENTS

The work described in this report was conducted under the general supervision of E.A. Finney, Director of the Research Laboratory Division, Michigan State Highway Department. The Laboratory is a Division of the Department's Office of Testing and Research, headed by W.W. McLaughlin, Testing and Research Engineer. The detailed design and layout of the experimental pavement was under the direction of H. Cash, Assistant Engineer of Bridge and Road Design.

The author expresses his gratitude to associates at the Laboratory for their able assistance, in particular to P. Milliman and his staff, who were responsible for the strain gage instrumentation and placement of the instrumented steel reinforcement, and to the project engineer and contractors' personnel for their fine assistance and cooperation.

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Appendix A

RECOMMENDATIONS ON MINIMUM REQUIREMENTS FOR TESTS AND OBSERVATIONS*

The following recommendations are made regarding minimum requirements for tests and observations:

A. During Construction:

1. Range in air temperature for construction period of each test section. (Obtained from a nearby weather station.)
2. Record of the mid-depth temperature of concrete at intervals of 500 ft. (Obtained just prior to final finishing.)
3. Mechanical tests on soil samples taken from the finished grade at intervals of not more than 1,000 ft, and where the subgrade soil changes in type. (Includes both the subbase material and subgrade soil.)
4. Concrete data to consist of information obtained by standard tests of the state. (Includes type of aggregates, proportions and consistency of mix, and strength characteristics.)
5. Reinforcing steel data to consist of information obtained by standard tests of the state.
6. Notes by qualified personnel of any unusual conditions that may affect performance.

B. After Construction:

1. Annual range in air temperature in the vicinity of the test site. (Obtained from a nearby weather station.)
2. Observations of the general condition of the pavement, including photographic records of any significant developments. (Made each spring and fall.)
3. Intensive crack survey of 500 ft of pavement located in the central region of each test section. (Made about the 15th of September, December, March, and June during the first year after construction and each fall thereafter.)
4. Measurements at the pavement surface of the widths of at least four selected cracks in each of the following locations of each test section: (a) central region, (b) 400 to 500 ft from one end, and (c) immediately following a construction joint.

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*Highway Research Board Subcommittee on Continuously-Reinforced Concrete Pavements; abridged from HRB Correlation Service Circular 372, p. 8 (Nov. 1958).

5. Photographic coverage of the surface condition of the pavement at not less than three of the apparently wider cracks in each of the following locations of each test section: (a) central region, and (b) immediately following a construction joint. (Taken in the outside lane before the pavement is opened to traffic and each fall thereafter.)
6. Cores at not less than two of the wider cracks in each test section. (Obtained within 6 months after construction, and at intervals of not more than 5 yr thereafter. Only one core should be taken at a given crack.)
7. Measurements of the absolute longitudinal movements at the terminal ends of the pavement. (Initial position determined at the time of construction. Subsequent measurements obtained during the hottest and coldest part of each year thereafter.)
8. Measurements of the changes in width of the terminal joints. (Initial reading taken at the time of construction. Subsequent measurements obtained during the hottest and coldest part of each year thereafter.)
9. Surface roughness indices of each test section. (Obtained before the pavement is opened to traffic and at intervals of not more than 3 yr thereafter.)
10. Traffic counts and particularly axle-load weights and frequencies. (Obtained soon after the pavement is opened to traffic, and at intervals of not more than 3 yr thereafter.)
11. Pertinent observations and measurements on the state's standard pavement to provide proper comparative data. (Includes surface roughness indices.)

Appendix B

INSTRUMENTATION AND INSTALLATION DETAILS FOR INSTRUMENTED STEEL REINFORCEMENT

The following presentation describes the materials and instrumentation procedure for the steel reinforcement strain phase of the project. Included in this appendix are the characteristics of the strain gages, the gage installation procedure, temperature compensation and reference gages, and a description of the process of taking the various readings.

Strain Gage Characteristics

All the strain gages used in connection with the steel reinforcement instrumentation were Type A-12, SR-4 electrical resistance wire gages. Each has a gage factor of 2.08 ± 1 percent and a gage resistance of 120 ± 0.2 ohms. The nominal length of the A-12 gage is 1 in. with a trim width of ⅛ in.

Gage Installation Procedure

Prior to actual placement of the strain gages, each of the five longitudinal wires of both the continuous mesh and standard mesh reinforcement was sanded to a uniform diameter for a length of 2 in. at the midpoint of the wire. In the case of the five No. 5 deformed bars of the bar mat reinforcement, each bar was turned on a lathe to a uniform diameter for a length of 2 in. at the center of the bar. Each wire or bar had an A-12 gage bonded to the top and bottom surfaces so that the longitudinal and transverse centerlines of the gages were diametrically opposite each other. The two gages were then wired in series to give a 240-ohm active bridge arm. The actual steps involved in preparing the reinforcement were:

1. Wire or bar sanded, cleaned, and solder tinned.
2. Tinning coat sanded and cleaned.
3. Armstrong A-1 cement precoat applied.
4. Precoat sanded.
5. Gages bonded to surface with Armstrong A-1 cement.
6. Gages covered with Armstrong A-1 cement.
7. Belden No. 8404 lead wires attached and secured with silk thread.

8. Entire installation covered with Armstrong A-1 cement.
9. Entire installation wrapped with ¾-in. linen motor winding tape.
10. Entire installation covered with Armstrong A-1 cement.
11. Entire installation again wrapped with linen tape.
12. Coat of paraffin applied to entire installation.

The adhesive agent used in the installation was Armstrong A-1, made by the Armstrong Products Co. This adhesive consists of two components, an epoxy resin formulation with an inorganic filler, and an amin-type catalyst. In all cases, Activator E was used as the catalyst because of its longer pot life and shorter curing time with heat application.

Temperature Compensation and Reference Gages

Temperature compensation was effected by embedding a steel plate with Type A-12 strain gages at each of the three instrumented reinforcement locations. The plates were hot-rolled steel, ¼ by 2½ by 4 in., with four gages bonded to each and wired in series to give two 240-ohm bridge arms. Two compensating arms were used to insure against possible gage failure. After protecting and waterproofing the installation, each plate was encased in a ⅛-in. thick foam rubber box, and the entire block covered with a thick coating of paraffin. To differentiate strains caused by temperature changes from those due to other factors, and also to check the longtime zero drift of the measuring instrument, four Type A-12 strain gages were bonded to a piece of 96 percent silica glass known by the trade name "Vycor." This material has a coefficient of thermal expansion of 0.44×10^{-6} per deg F, and was mounted in the form of hollow tubes in a styrofoam insulated box to minimize the effects of any rapid temperature change. The gages were wired to provide two 240-ohm arms to check for the zero drift of the strain indicator.

Reading Procedure

All strain readings were taken with a Baldwin SR-4 static strain indicator. Each set of readings taken at an instrumented steel location consisted of the following:

1. Zero Drift.—(a) Vycor arm A active, Vycor arm B compensating; and (b) Vycor arm B active, Vycor arm A compensating.
2. Temperature Strain.—Reading of each of the temperature compensating gages as active, with Vycor arm A as compensating.
3. Total Strain.—Reading of the five sets of gages on the reinforcing steel as active, with Vycor arm A as compensating.
4. Total Strain Minus Temperature Strain.—Reading of each of the five sets of gages on the reinforcing steel as active, with each of the two temperature compensating gages as compensating.
5. Gages.—Reading of resistance to ground of all sets of gages.

Appendix C

CONSTRUCTION DATA AND MATERIALS CHARACTERISTICS

TABLE 1

DAILY AIR TEMPERATURE VARIATION

	Degrees F				Degrees F		
Date	High	Mean	Low	Date	High	Mean	Low
9-22-58	72	60	52	10-12-58	55	43	29
9-23	74	61	49	10-13	67	51	42
9-24	81	70	60	10-14	70	58	44
9-25	77	67	54	10-15	72	61	47
9-26	66	56	49	10-16	69	60	50
9-27	65	53	42	10-17	60	48	37
9-28	63	47	37	10-18	54	42	33
9-29	62	50	38	10-19	59	45	33
9-30	50	45	37	10-20	62	48	34
10-1	49	39	32	10-21	69	53	39
10-2	59	45	35	10-22	76	54	45
10-3	62	48	37	10-23	58	52	43
10-4	73	57	44	10-24	48	44	40
10-5	50	41	31	10-25	57	48	44
10-6	62	47	30	10-26	49	44	36
10-7	76	62	51	10-27	50	41	36
10-8	69	60	55	10-28	50	39	32
10-9	70	61	54	10-29	52	38	29
10-10	58	47	43	10-30	55	41	31
10-11	48	41	32	10-31	60	46	39

TABLE 2

CONSTRUCTION DATA

				Modulus of Rupture (psi)		Concrete Temperature (deg F)			Air Temperature (deg F)			Compressive Strength[1]	
Pour Date	Roadway	Pour Length (ft)	Stationing	7 Day	28 Day	High	Avg.	Low	High	Avg.	Low	No. of Samples	Avg. Strength (psi)
9-22-58	Eastbound	671	838+25 to 844+96	550	650	77	73	68	72	65	58	1	4770
9-23	Eastbound	2110	844+96 to 866+06	650	750	81	77	72	74	71	64	2	5390
9-25	Eastbound	2411	866+06 to 890+17	525	575	80	77	75	77	70	61	2	5000
9-26	Eastbound	2343	890+17 to 913+60	590	700	76	74	69	66	60	52	2	4870
9-27	Eastbound	2332	913+60 to 936+92	660	780	75	71	66	65	58	50	3	5290
9-29	Eastbound	2313	936+92 to 960+05	760	788	70	65	58	63	55	42	2	5370
9-30	Eastbound	240	960+05 to 962+45	590	None	66	65	64	50	50	50	None	-
10-2	Eastbound	2755	962+45 to 990+00	576[2]	930[2]	66	62	50	60	51	35	3	5180
10-3	Eastbound	2740	990+00 to 1017+40	610	800[2]	70	63	54	62	51	37	3	5750
10-4	Eastbound	2720	1017+40 to 1044+60	720	740	74	67	60	73	64	47	3	5510
10-6	Eastbound	3174	1044+60 to 1076+34	584	925	66	62	52	62	52	33	3	6000
10-7	Eastbound	2756	1076+34 to 1103+90	737	928	77	74	62	76	68	51	3	5670
10-8	Eastbound	295	1103+90 to 1106+85	596[2]	None	69	69	69	58	57	56	None	-
10-13	Westbound	3150	1080+00 to 1048+50	437	688	70	66	58	67	57	43	3	4550
10-14	Westbound	2950	1048+50 to 1021+00	550	675[2]	76	71	64	70	63	50	3	5070
10-15	Westbound	3100	1021+00 to 990+00	583	798	88	82	78	72	65	50	3	4920
10-16	Westbound	2396	990+00 to 966+04	575[2]	775[2]	84	76	71	69	65	55	2	4900
10-17	Westbound	3504	966+04 to 931+00	561	676	71	66	58	60	52	40	4	5330
10-18	Westbound	3450	931+00 to 896+50	523	729	70	69	66	54	48	33	3	5320
10-20	Westbound	3277	896+50 to 863+73	482	709	68	62	55	62	54	37	3	5280

[1] As determined from cores taken on 12-11-58, and tested 4-15-59.
[2] Denotes only one specimen available.

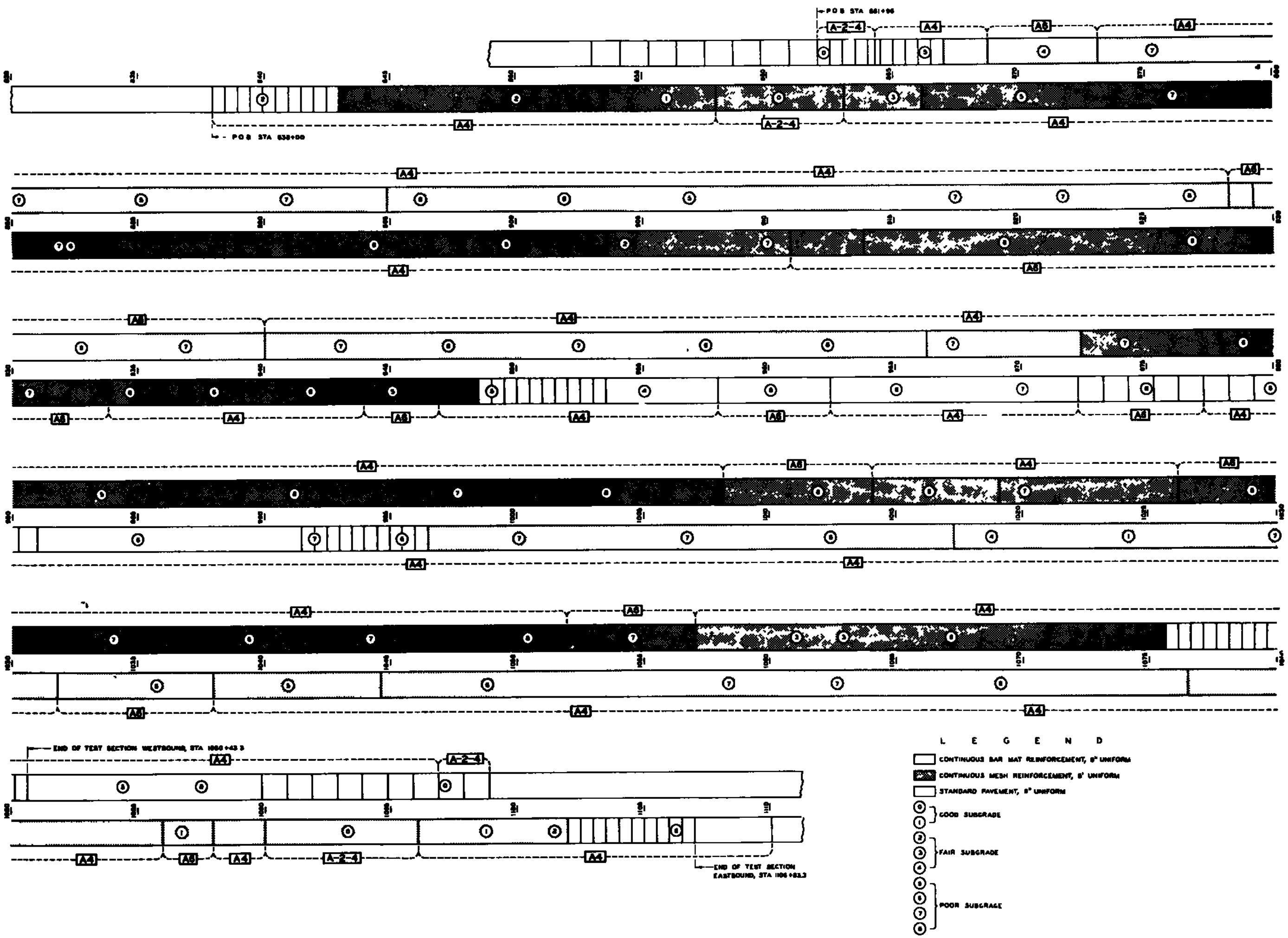
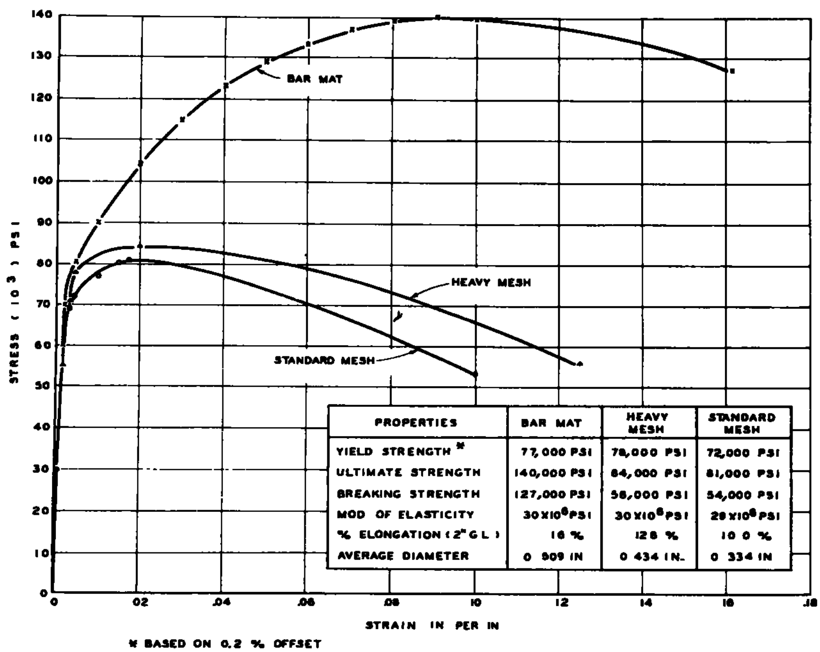


Figure 49. Subgrade soil classification (AASHO classification system).



PROPERTIES	BAR MAT	HEAVY MESH	STANDARD MESH
YIELD STRENGTH *	77,000 PSI	78,000 PSI	72,000 PSI
ULTIMATE STRENGTH	140,000 PSI	84,000 PSI	81,000 PSI
BREAKING STRENGTH	127,000 PSI	58,000 PSI	54,000 PSI
MOD OF ELASTICITY	30×10^6 PSI	30×10^6 PSI	29×10^6 PSI
% ELONGATION (2" G L)	16 %	12 8 %	10 0 %
AVERAGE DIAMETER	0 909 IN	0 434 IN.	0 334 IN

Figure 50. Steel reinforcement properties.

The Problem of Corrosion of Load Transfer Dowels

ROBERT G. MITCHELL, Chief, Planning-Traffic-Design, Connecticut State Highway Department

The effect of corrosion on performance of load transfer dowels in transverse expansion joints in concrete pavements has been a subject of some concern for a number of years. This paper constitutes a report on the observations of dowel performance in transverse expansion joints on Connecticut pavements of an earlier design; that is, 100-ft slab length with three intermediate warping joints. It describes the condition of different types of hot-rolled dowels and beams removed from pavements of varying ages.

In a further attempt to evaluate various modern, rust-resistant coatings and claddings, standard abrasion and salt spray tests were performed at a commercial laboratory. Those materials with apparent promise were used in actual field installations and are under observation to determine their effectiveness in preventing seizure due to corrosion. These installations are in pavements of the more recently adopted 40-ft contraction joint design. Similar rust-resistant dowels were partially cast in concrete and submitted to accelerated test in a tidal area. The conditions pertaining in the various test installations in late fall 1959 are described.

●JOINTS in concrete pavement, and particularly certain troublesome features of joints, have been the subject of considerable investigation and study. The introduction of load transfer dowels helped to correct some earlier problems while creating new ones.

The effect of corrosion on the performance of load transfer dowels in transverse expansion joints in concrete pavements is not a new subject and has been of some concern for a number of years. An outstanding treatise on the subject is that by William Van Breemen "Experimental Dowel Installations in New Jersey," HRB Proc., Vol. 34, p. 8 (1955). Although less concerned with expansion joints, corrosion as an influence on dowel performance is also brought to attention in the paper by Ernest T. Perkins, "Test Project Constructed Utilizing the Contraction Joint Design," HRB Bull. 165, p. 35.

Further investigation by Connecticut of dowel corrosion resulted from observations made in July 1956 during an inspection of some pavements with Bengt. Friberg, Consulting Engineer. On the eastbound roadway of Route 15 in Manchester, a rather consistent, excessive opening of the transverse, weakened plane joint at the mid-point of the slabs was noted. These weakened plane joints are sometimes called "hinged" or "warping joints" as the pavement reinforcement is not interrupted at these joints. They will hereinafter be referred to only as "intermediate joints."

In each case, the joint seal at the expansion joints appeared to be in an undisturbed condition, whereas the seal in the open intermediate joints was quite obviously in very poor condition. This seemed to indicate that slab movement, due to temperature changes, was occurring mainly at the intermediate joints, and that little or no movement was taking place at the expansion joints. Removal of a portion of the shoulder at one of the open intermediate joints revealed that the reinforcing steel through the joint was probably broken and the joint was now functioning as a contraction joint instead of as a tightly held crack. This pavement had been placed in 1948.

In September 1956, during an inspection of the concrete pavement of the eastbound roadway on Route 15 in the Town of Union, it was noted that several of the sawed intermediate joints were open from ½ in. to ¾ in. A general survey of this pavement, which was constructed in 1953, indicated a prevalence in the opening of the intermediate joints at the center of the slabs.

The pavement design at both of these locations was essentially the same. The expansion joint spacing was approximately 100 ft, with either formed or sawed intermediate joints spaced at approximately 25-ft intevals. Fabric reinforcement equal to 0.135 percent of the concrete area and weighing 61 lb per 100 sq ft was carried through the intermediate joints.

Random surveys, subsequently made, of concrete pavements constructed to similar design standards revealed the excessive opening of the intermediate joints to be quite prevalent. Because, in many cases, the joint seal in the expansion joints appeared in such exceptionally good condition, it was suspected that the load transfer dowels were either totally or partially restraining movement at the slab ends, with the result that compensating movement was taking place at the intermediate joints. In many instances, heavy trucks had caused the leading edge of the pavement at the intermediate joint to fault considerably, so that mudjacking was eventually required at locations of severe faulting.

PROCEDURE AND OBSERVATIONS

It was decided to cut out a portion of the concrete pavement containing one load transfer dowel at each of three expansion joints and a similar section of the concrete pavement across each of two wide open intermediate joints in the eastbound roadway at Route 15 in the Town of Manchester. A similar number of samples were also to be cut out of the eastbound roadway of Route 15 in the Town of Union. The purpose was to determine the condition of the dowels in the pavement, and whether the amount of restraint offered by the dowels to movement at the expansion joints might be the cause of the excessive opening of the intermediate joints.

Field

Figure 1 shows the location of the joints and the pavement areas cut out in the Towns of Manchester and Union. In October 1956, test blocks labeled 1M, 3M and 4M, were cut from the expansion joints and specimens labeled 2M and 5M from the open formed intermediate joints at the left edge of the inside lane of the eastbound roadway in the Town of Manchester. At the same time test blocks labeled 1U, 3U and 5U were cut from the expansion joints and two specimens labeled 2U and 4U from the sawed intermediate joints along the outside edge of the travel lane of the eastbound roadway in the Town of Union.

The use of a 22-in. concrete saw in cutting the 24- by 12-in. blocks from the 8-in. pavement resulted in some necessary over-cutting (Fig. 2). Areas cut out were backfilled with a cold bituminous mixture, but heavy truck traffic did cause cracking of pavement at the corners of pavement where over-cutting occurred.

Figure 3 shows expansion joints 1M, 3M and 4M on Route 15, Manchester, at which test blocks were cut out. The joint seal at all three joints is in very good condition with no indication of any adhesive or cohesive failures. A cross-section of expansion joint 4M showing the joint filler and the metal shield enclosing the filler as they appeared after 8 years of service is shown in Figure 4. The original ½-in. joint filler installed in this pavement in 1948 has been compressed to ⅛ in. The compressed condition of the joint filler is undoubtedly typical of a large number of expansion joints in concrete pavements of this design. Observations of ½-in. cork filler removed from the expansion joints on the Merritt Parkway in 1945 indicated a similar state of extreme compression along with frozen dowels.

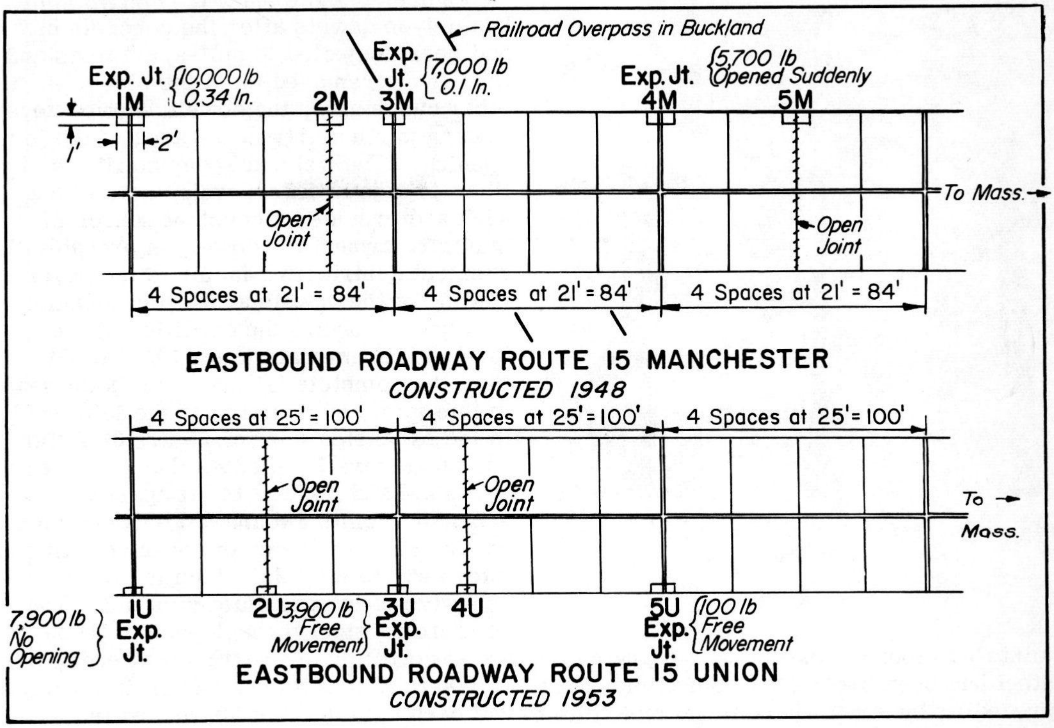


Figure 1. Location of joint cutouts.

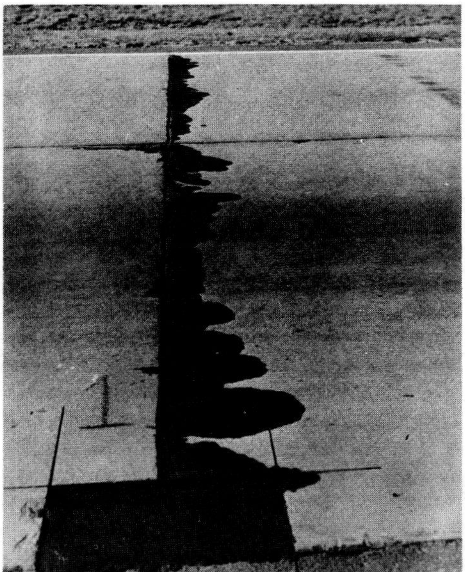


Figure 2. Expansion joints 1U and 5U on Rt. 15 in Union.

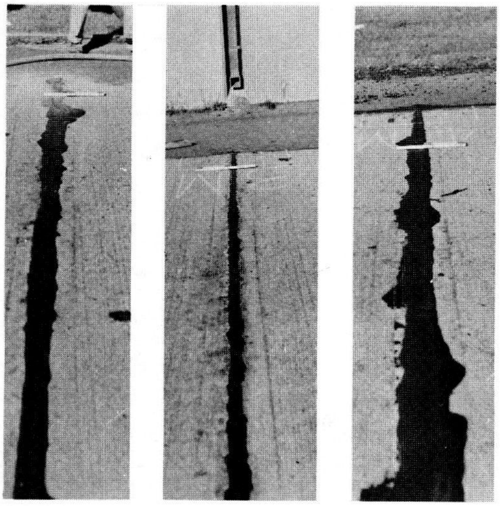


Figure 3. Expansion joints 1M, 3M and 4M on Rt. 15 in Manchester.

Figures 5 and 6 show the condition of beam-type dowels after the concrete blocks had been subjected to pull-apart tests and the dowels removed from the block. In all other cases also, the dowels showed severe rusting particularly in the area of the joint opening. The wet and soggy condition of the fiber-type joint filler, which acts like a wick and provides a constant source of moisture around the dowel, is probably the greatest contributing factor to the severe rusting of the dowels at the joint opening.

Figure 7 shows the condition of the formed intermediate joints 2M and 5M. Note the complete failure of the joint seal in this joint as compared to the joint seal in the expansion joint of Figure 3. Reference to Figure 1 indicates that intermediate joints 2M and 5M are 147 ft apart with two expansion joints 3M and 4M between them. Figure 8 shows the extent of the opening of intermediate joint 2M which is also representative of intermediate joint 5M. The reinforcing steel through each of these joints was found broken and, judging by the rusted condition of the fractured ends, the steel had been broken for some time. In the 1, 000 ft of concrete pavement immediately preceding the area where the sample blocks had been cut out, 7 of 30 formed intermediate joints have opened abnormally in either one or both lanes.

In Figure 9 the ends of the broken reinforcing steel may, perhaps, be seen with the aid of a magnifying glass. The extent to which the sealing material has flowed down into the joint may also be observed.

Figure 2, previously discussed, shows the well-sealed condition of expansion joint 1U on Route 15, Union, at which a test block was removed. At expansion joint 5U there was evidence of considerable bond or adhesive failure. In the pull-apart tests, a load

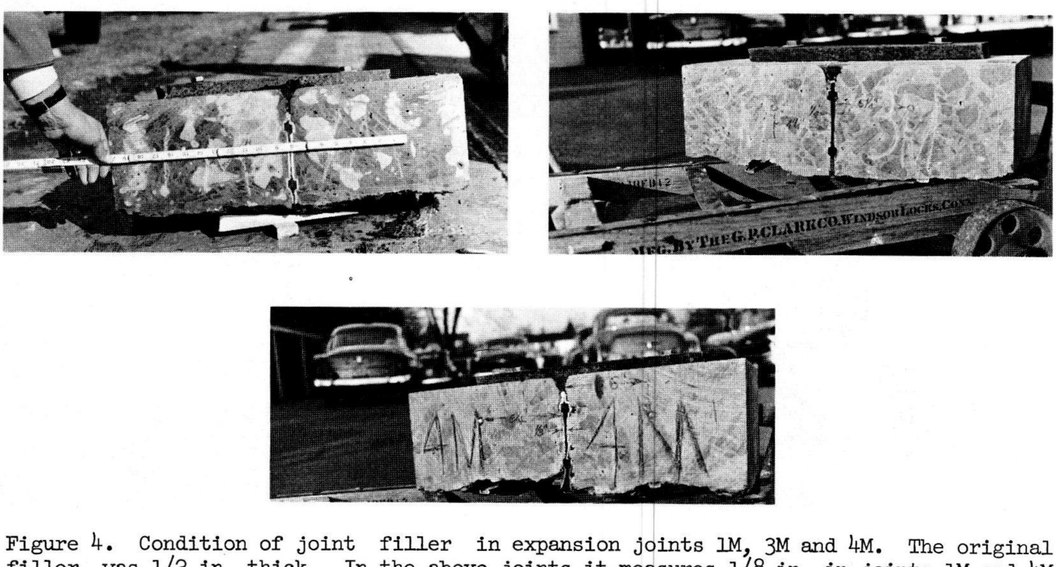


Figure 4. Condition of joint filler in expansion joints 1M, 3M and 4M. The original filler was 1/2 in. thick. In the above joints it measures 1/8 in. in joints 1M and 4M and 1/4 in. in joint 3M.

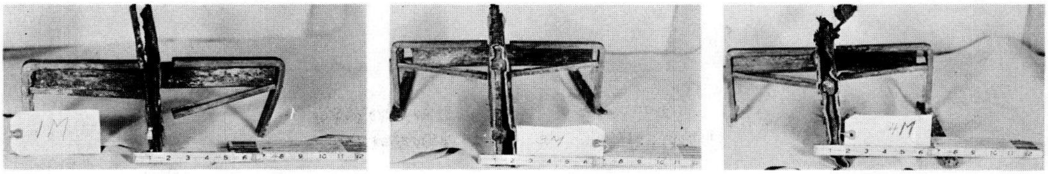


Figure 5. Condition of load transfer dowels in expansion joints 1M, 3M and 4M.

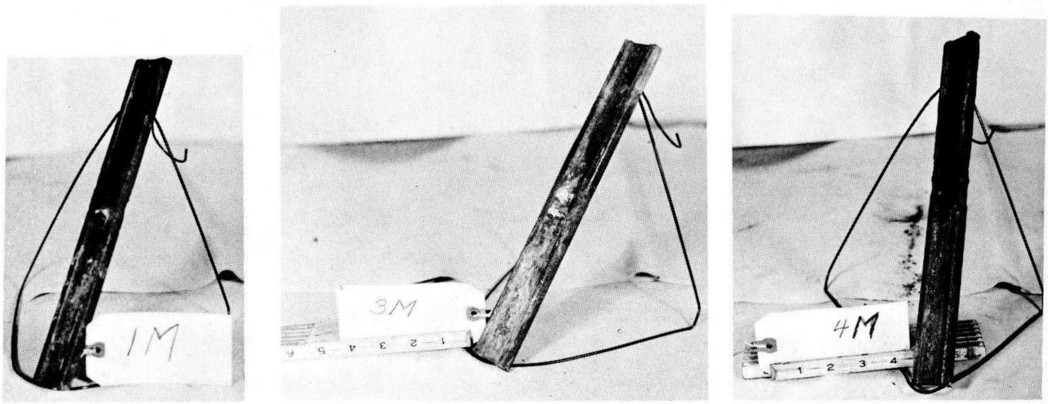


Figure 6. Rust and erosion of dowels from expansion joints 1M, 3M and 4M.

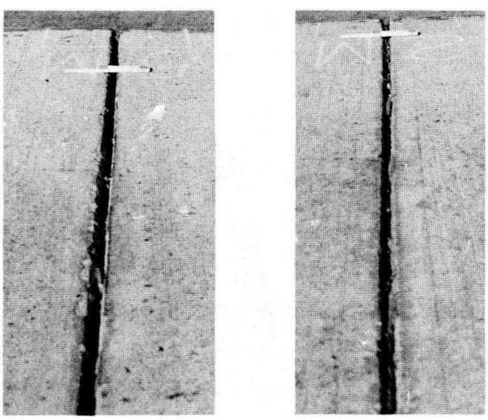


Figure 7. Condition of joint seal and opening of intermediate formed joints 2M and 5M on Rt. 15 in Manchester.

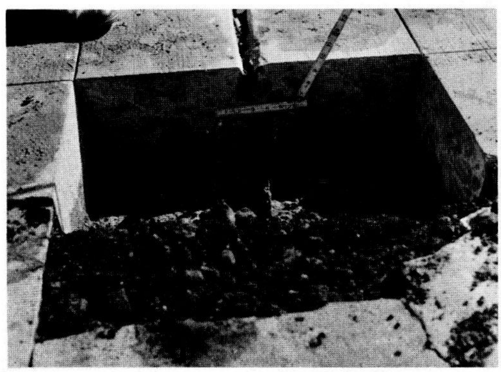


Figure 8. Width of opening at intermediate joint 2M. The longitudinal steel of the reinforcing mat is broken and joint has opened 5/8 in.

of 7,900 lb did not cause any movement at expansion joint 1U, but at joint 5U a load of 100 lb pulled this joint completely apart. It is quite evident, therefore, that in judging the merits of a joint seal in any expansion joint on the basis of appearance it is important to ascertain whether or not the joint is functioning properly.

The original ¾-in. non-extruding-type joint filler installed in this pavement in 1953 has, after 3 years of service, been compressed to ½ in. as indicated in Figure 10.

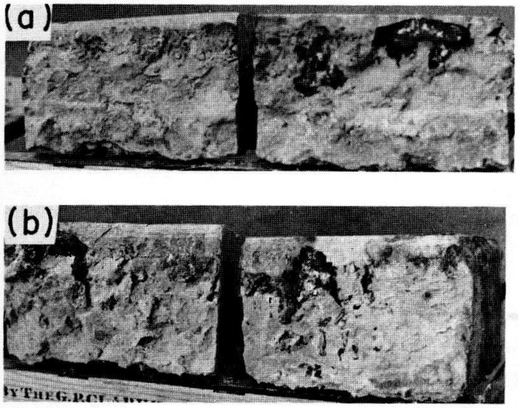


Figure 9. Joint faces and the ends of the broken longitudinal steel—(a) intermediate formed joint 2M and (b) intermediate formed joint 5M.

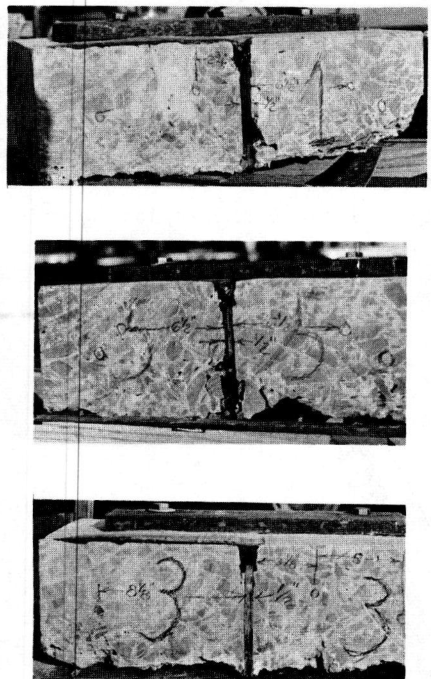


Figure 10. Expansion joint filler originally 3/4 in. non-extruding fiber filler. It now measures 1/2 in.

The condition of round dowel 1U (Fig. 11) after removal from the test block indicates considerable rusting at the joint opening decreasing in severity along the sliding end of the dowel. These dowels were plain round steel dowels with no coatings, except for greasing of the sliding end. Except where concrete was noted to be porous, the fixed end of the dowel shows no indication of rust progressing along this end. Here again the water-soaked joint filler is probably responsible to a large

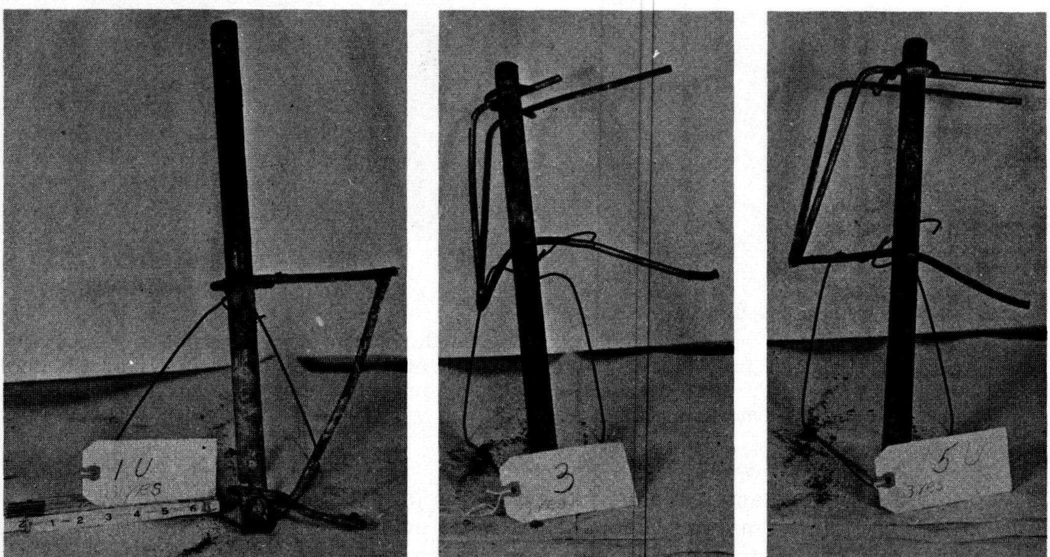


Figure 11. View of dowels after removal from concrete. Some rust has formed at the opening. The dowels were wire brushed to determine loss of section due to rust.

extent for the formation of rust on the sliding portion of this type of dowel.

Mention was previously made of the fact that it required a load of only 100 lb to open the expansion joint of test block 5U. Figure 12 shows why this small load opened this joint completely. The longitudinal section of the concrete pavement encasing the sliding end of the dowel is a good example of poor consolidation of the concrete around the dowel and lack of intimate contact essential to an efficient transfer of load across the joint. This fact may account for the occasional faulting noted in the expansion joints throughout this project. From the appearance of the joint faces of specimens 1U and 5U (Fig. 12) the wire assembly holding the dowel tends to prevent thorough encasement of the entire dowel assembly by the concrete.

Figure 13 shows the wide open condition of the sawed intermediate joints 2U and 4U. The respective locations of sawed intermediate joints and the expansion joints involved are shown in Figure 1 on the eastbound roadway of Route 15, Union. Joint 4U in Figure 13 is interesting because it definitely shows that in two lanes of concrete, with longitudinal tie bars on 5-ft centers, there can be limited movement in one lane without affecting the adjacent lane. The sawed joint in the far lane appears to have undergone no change in width whereas the sawed joint in the near lane has opened to ¾ in. There is, however, some evidence of distress along the longitudinal joint adjacent to the transverse expansion joint due to the differential lane movement. On removal of these test blocks from the pavement, it was found that, at the time of construction, the longitudinal reinforcing steel had been sawed completely through at joint 2U and 75 percent through the steel in joint 4U. While this was true in the aforementioned two intermediate joints, it has been observed in two cases, where other sawed joints pulled apart on this project, that the steel appeared to be broken rather than sawed. In the latter two cases no test blocks were removed from the pavement but the open joint was blown out with compressed air, and the ends of the steel reinforcement were observed in the open joint with the aid of a flashlight.

Inspection of 1,300 ft of pavement, including the area in which test blocks were removed from both the expansion joints and the sawed intermediate joints, disclosed that 10 sawed joints of 39 had opened abnormally.

Basically, the concrete pavement design used throughout Route 15 provided for expansion joints at approximately 100-ft intervals and either formed or sawed intermediate joints at approximately 25-ft intervals with the reinforcing steel carried through the intermediate joints. These intermediate joints are in effect predetermined cracks in the concrete pavement and, as such, they should be held tightly together.

The concrete slabs in the eastbound roadway of a test road constructed in 1948 at Portland, Conn., varied in length between expansion joints from 96 ft to 172 ft. There were no intermediate joints in this pavement and the reinforcing steel, which was of the same weight and cross-section as that used on Route 15, was held constant throughout. In October 1950, one crack appeared in the 154-ft slab. To the end of 1956, no further cracking was noted in any of the other slabs. When first observed, this crack was very fine and appeared to end at a point

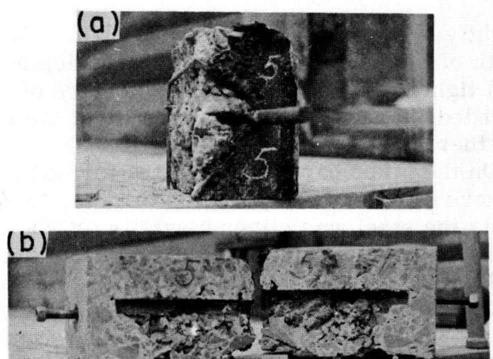


Figure 12. Upper photos (a) and (b) show poor consolidation of concrete around the dowel slip end from joint 5U. Lower photo (c) shows poor consolidation of concrete around the fixed dowel end at joint 1U.

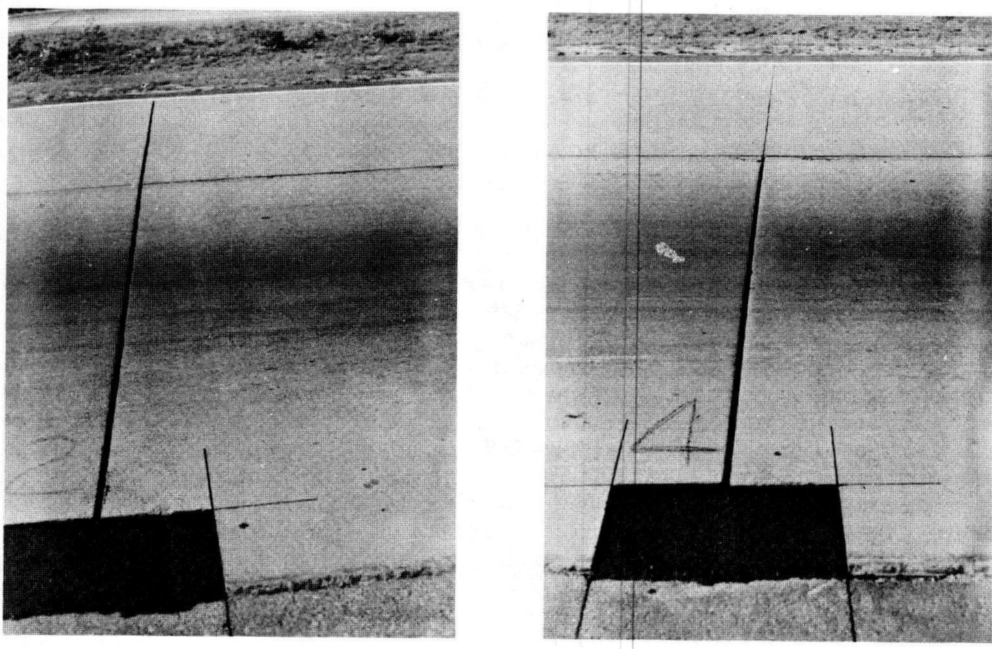


Figure 13. Intermediate sawed joints on Rt. 15 in Union. Note comparative widths of sawed joint #4 across the pavement. Joints were sawed originally full width of pavement.

slightly over half-way across the slab. This crack now is plainly visible for the full width of the slab and appears to be widening slightly, although the crack faces are still held tightly together. After eight years of service this pavement is still functioning as intended. One crack has appeared but the reinforcing steel is holding the faces tightly together.

On the other hand, many of the formed and the sawed intermediate joints on Route 15 have cracked and opened up to $\frac{1}{4}$ in. to $\frac{3}{4}$ in. Undoubtedly in a number of these joints the steel was either partially or wholly sawed through, in the case of the sawed joints, which accounts for the numerous excessive intermediate joint openings at the early age of 3 years. Where the intermediate joints were formed, however, there is no possibility of the steel being prematurely weakened by sawing. Although no information is available as to when the formed joints opened and the reinforcing steel broke, it seems reasonable to assume that this condition occurred at a much later date than in the case of the sawed joints.

Joint movement data obtained on the Vernon test section on Route 15, constructed in 1952, showed, at the end of three years of service, that 15 sawed intermediate joints had opened an average of 0.015 in. Two of the 15 joints observed had each opened 0.030 and 0.022 in., respectively. At the end of 4 years of service the average opening of the 15 sawed intermediate joints was 0.033 in., the aforementioned two joints had each opened to 0.12 in. and a third joint had opened to 0.04 in. In January 1959, the average opening of the 15 intermediate joints was 0.12 in. The two aforementioned joints were opened 0.604 in. and 0.426 in. and the third joint was opened 0.463 in. Very obvious faulting is occurring at these three joints. Movement at the expansion joints nearest to these intermediate joints has become very small due either to dowel restraint, or to shortening of slab lengths due to breaking of steel reinforcement, or a combination of both. On the basis of this data it appears that some of the sawed joints may be expected to open appreciably after three years of service.

Based on tests conducted by the Bureau of Public Roads, Sutherland and Cashell have stated that effective stress control due to aggregate interlock at a joint is not dependable when the joints are open 0.04 in., or more, irrespective of the type of aggregate used. Therefore, at the intermediate joints with openings of 0.10 in., or more, there are in effect free pavement ends. Due to the deflection of these free ends under heavy loads, the flexural stresses plus the tensile stress due to reduction of pavement temperature may become sufficient to break the steel completely.

The wire mesh reinforcement removed from the test blocks cut out in both Manchester and Union on Route 15 was checked by the laboratory for compliance with the standards and specifications applying at the time of construction of this pavement. The steel was found to comply in all respects with the requirements at that time. The range in breaking strength of the steel tested varied from 75,050 psi to 84,280 psi. The minimum tensile strength allowable per ASTM A82 is 70,000 psi with the yield point as 0.8 the tensile strength. Assuming the average tensile strength of the wire mesh as used on Route 15 was 80,000 psi, the yield strength would be 0.8 of 80,000 psi or 64,000 psi. Since the area of steel reinforcement for a 12-ft lane is 0.135 percent of the concrete area, the load required to develop the unit stress of 64,000 psi is equal to 8,320 lb per foot of width of concrete pavement. In a concrete slab 100 ft in length, and with no restraint to movement other than subgrade friction, the pull developed at the midpoint of the slab during a period of contraction of the concrete might very well be 5,000 lb per foot of width of concrete, assuming a coefficient of friction of 1.0. These figures are considered quite conservative. For the foregoing conditions, the stress developed in the steel, assuming the concrete had cracked, would be well below the actual yield point of the steel and any cracks which might occur should, under these conditions, remain tightly closed. As will be shown, however, the ends of the concrete slab are restrained from moving due either to rusting of the dowels, misalignment or a combination of both. With dowel restraint of such magnitude as to require a pull of 3,000 lb per foot of pavement to open the joint 0.1 in. (which is equal to 20 F change in pavement temperature) the total load placed on the steel, assuming the concrete has cracked, is now 8,000 lb. This leaves a very small margin to reach the yield point of 8,320 lb, which might very well be exceeded when a heavy truck passes over the crack and causes a slight flexing of the steel.

Laboratory

To determine the extent of the dowel restraint, the concrete blocks each containing one dowel were set up in a laboratory testing machine. A load was applied to the ends of the concrete through a steel yoke attached to the concrete with 5/8-in. steel bolts. Lead wool was compacted around the bolts in the concrete. This method of holding the bolts in the concrete was not very satisfactory as the bolts tended to pull out under loads over 5,000 lb. Two Ames dial gages were attached to the concrete at the joint in order to read the amount of opening as the load was applied. Unfortunately, the joint openings under load were too erratic to obtain the load required for any particular measured amount of opening, because in some cases, considerable load might be applied and no appreciable movement occur until the dowel restraint was overcome and then considerable movement might occur. The results of the pull-apart tests on the blocks removed in Manchester, and which contained the beam-type dowel, were as follows:

1. A load of 1,500 lb was required to open joint 1M 0.1 in. and a load of 10,000 lb to open the joint 0.34 in. An opening of 0.1 in. is equivalent to a change of 20 F in pavement temperature and an opening of 0.34 in. is equal to a change of 55 F in pavement temperature. Maximum recorded seasonal changes in pavement temperatures vary from 80 F to 100 F.
2. A load of 7,000 lb was required to open expansion joint 3M 0.1 in. and further loading had to be suspended due to cracking of concrete around the yoke bolts.
3. Joint 4M opened suddenly after a load of 5,700 lb had been applied. The load was increased up to 8,300 lb with a joint opening of 0.52 in.

The following are the results of the pull-apart tests on the expansion joints removed in Union and which contained the round-type dowel.

1. Joint 1U would not open under a load of 7,900 lb. Further loading was not applied because the yoke bolts began to pull out.

2. A load of 3,700 lb opened joint 3U 0.1 in. and at 3,900 lb the dowel pulled out of the concrete.

3. Joint 5U practically fell apart under a load of 100 lb. As previously mentioned this was due to poor consolidation of concrete around the dowel.

Discussion

Although it is improbable that the test results shown for single load transfer dowels apply equally to all the dowels in the complete load transfer assembly of a lane, it is desirable to project these test results for the full joint length, even though somewhat questionable assumptions are made.

In a 12-ft lane of 8-in. concrete pavement there are installed 10 beam-type dowels or 12 of the round-type dowels. On the basis of the data obtained in the pull-apart tests, restraint to movement offered by either type of dowel may very well require an average pull of 5,000 lb per dowel to open a joint 0.1 in. For the beam-type dowels the total pull in a 12-ft lane would be 50,000 lb or for the round-type dowel 60,000 lb. This is equal to 4,167 lb and 5,000 lb, respectively, per foot of width of concrete. Assuming that the point of zero movement due to temperature change is at the center of the slab, the weight of a longitudinal 1-ft strip of concrete with an expansion joint spacing of 100 ft, which must be overcome during a period of contraction of the concrete, is equal to 5,000 lb. The total force required to move the concrete 0.1 in. would then be 5,000 lb plus 4,167 lb or 9,167 lb per foot of width of concrete for the beam-type dowel, and 10,000 lb per foot of width of concrete for the round dowel. As previously stated, this force combined with tensile stress induced by heavy loads during the winter months is sufficient to break the longitudinal reinforcing steel. Where the steel was partially cut in the process of sawing the intermediate joints, a considerably lower force could break the reinforcement at an earlier age.

In summary the following facts are cited:

1. The intermediate joints in those pavements which provide an expansion joint spacing of approximately 99 ft to 100 ft are cracking and are gradually opening from ¼ in. to ¾ in. despite the use of reinforcement through the joint which should maintain a tight crack.

2. In general this condition occurs at the central intermediate joint, although there are instances where the joint adjacent to the expansion joint has opened considerably rather than the central joint.

3. The reinforcement through the formed intermediate joints is broken, and plainly visible joint faulting is occurring. The reinforcement through the sawed intermediate joints had been either partially or totally cut during the process of sawing the joints. Based on data obtained of the movement at the sawed joints on the Vernon test section, an opening of ⅛ in. was observed at two of the sawed joints at the end of three years. Variations in the degree of joint opening are probably related to the extent to which the steel was cut during the sawing of joints, and dowel restraint to movement.

4. There is considerable restraint to movement at the joint, due, it is believed, to rust on the dowels at the joint opening which tends to progress along the sliding end of the dowels. Based on the data obtained in the pull-apart tests of small joint sections cut out of the pavement, this restraint varies from 3,000 lb to over 8,000 lb per dowel. The joint filler retains the moisture to a very large degree and maintains a continuously wet condition around the dowel across the joint.

5. Notwithstanding the evidence herein, the long slabs without intermediate joints constructed on the Portland test road in 1948 have, after 8 years of service, developed but one single crack which is, however, held tightly together. The same unit area of steel reinforcement per foot of width of concrete was installed in this pavement as was used in the concrete pavement of Route 15.

In view of these facts it is concluded that the degree of restrained movement due to dowel corrosion is sufficient to cause the reinforcing steel to break and the intermediate joints to open much wider than was anticipated. Due to failure of the seal in these joints, sand and other incompressible materials have infiltrated into these joints and caused them to remain open. Consequently, during period of hot weather the expansion joints have been subjected to considerable compressive force which has gradually reduced the thickness and effectiveness of the joint filler. While the partially sawed steel is responsible for more frequent and wider opening of the intermediate joints at an early age, it is quite apparent that this condition will eventually prevail even where these joints are formed. This condition appears mainly in the pavement design which provides for an expansion joint spacing of 99 to 100 ft with equally spaced intermediate joints.

EXPERIMENTAL WORK

Early reports on the foregoing observations and tests led to the decision to explore the possibilities of rust resistant dowels. For round dowels, sleeves of rust resistant metals constituted a possible solution. For the beam-type dowel, some other means of applying a protective coating seemed indicated, and some of the more modern materials in this line held promise.

In early 1957, the cooperation of a number of manufacturers of dowels and coating materials was obtained. Time for testing was restricted by the decision to make experimental dowel installations on the Connecticut Turnpike, which was then in part in the grading stage.

Laboratory

Arrangements were made with a commercial laboratory to proceed with a salt spray test on specimens submitted, to be followed by Taber abrasion test on standard specimen plates coated with the more promising protective coatings. While there might be a question as to the choice of tests, they had the advantage of being standard tests which could be completed during the available time.

Submitted for the salt spray test were 21 specimens composed of 19 round dowels and beam dowels protected with 12 different coatings, one round dowel partly sleeved with stainless steel, one 431 heat treated stainless steel dowel beam and one each Parkerized and plain untreated dowel beams.

The conditions of test were reported as follows:

PH	6.9
Temperature	95 F
Specific gravity of solution	1.140
Fog collected (ml/hr)	0.65

Observations were made at 24-hr intervals and the test was stopped after 2,088 hr. Some of the specimens were withdrawn before the completion of the test because advanced rust formation was evident. Early rusting was observed on all unprotected steel surfaces and on the areas exposed by an X scratched through the coatings.

The Taber abrasion test was performed on plates representing 11 of the coatings which showed apparent promise, of these only five withstood 4,000 cycles.

There was concern about the ability of these coatings to withstand rough handling during construction. A crude, manual impact test eliminated another coating which demonstrated extremely brittle characteristics.

Field Installations

On the basis of these tests the four coatings, which are patented commercial processes, showing best performance were selected for application on beam dowel assemblies to be installed on the Connecticut Turnpike in Greenwich. As it were, a 40-ft contraction joint design had been selected for this pavement in preference to the 100-ft slab with intermediate joints.

Sufficient beam dowels were coated to provide for four groups of 25 contraction joint assemblies for the three 12-ft lanes of 9- and 10- in. pavement. Bond breaking grease was applied to the sliding end of all dowels prior to shipment to the job site. Installation of the experimental dowels was completed by November 1957. Plugs for measurement of joint movement were installed at the shoulder edge of the pavement at all joints containing experimental beam dowels and in a control section of 25 joints with untreated beam dowels.

In May 1957, a considerable number of contraction joint load transfer assemblies with round dowels partly sleeved with stainless steel were installed on the Connecticut Turnpike, Milford. Here also plugs were placed at 25 joints each of the sleeved and regular dowels.

At a third location on the Connecticut Turnpike, Old Lyme, load transfer assemblies containing round dowels, partly sleeved with Monel metal, were installed in twelve 24-ft contraction joints in May 1958.

A fourth installation comprises 12 contraction joints, 24 ft long, which contain load transfer assemblies with round steel dowels, nickel coated or clad for their full length. This installation was completed on Route 9, Middletown, in August 1958. On these last two installations plugs were also placed at the experimental joints and at control joints.

On the basis of measurement of movement occurring at the joints containing experimental as well as control load transfer assemblies, there is, to date, no marked difference or trend in the performance as far as unrestrained movement is concerned.

Accelerated Test

When the load transfer assemblies containing the variously coated beam dowels were manufactured, it was requested that additional coated beam dowels be furnished for further tests. So far the only other test attempted consists of exposure of such dowels to tidal waters. For this purpose, three each of the beam dowels protected with the four different coatings, two monel sleeved round dowels, and three nickel clad round dowels were partly encased in concrete in "dumb-bell" fashion. A piece of reinforcing rod was cast in, parallel to the dowels, so that the assemblies could be lined up on a supporting rod to prevent casual removal. These dowels were placed in the tidal range of the Black River at Old Saybrook on April 16, 1959.

The dowels were examined on September 24, 1959 after five months of exposure to the salt water. The nickel coated dowels showed no signs of rust. The two monel clad dowels also showed no signs of rust although there is evidence of slight pitting of the metal sleeve. The remaining dowels, which are of the beam type and were coated with various corrosion resistant materials, show very slight signs of rust. One of three dowels coated with the same material did show about 1 sq in. of rust on the web whereas the remaining two showed only tiny spots.

In general the steel channels in which the beam-type dowels slide and which were also coated with corrosion resistant material, show slight signs of rusting at the outside corner edges of the channels. It appears that the coating may not be sufficiently thick at these 90-deg corners.

At the present time the nickel coated dowel appears to hold considerable promise of a rust free dowel. Any conclusive evidence of a superior rust resisting quality between the various coatings used has not been noticed at this time.

Radiography

There has been some speculation as to what extent, if any, radiography might help in the detection of dowel corrosion. In turn, there is a question whether or not radiography can distinguish between steel and iron oxide unless there were a considerable loss of section. The Department was fortunate in obtaining the cooperation of a manufacturer of radiographic equipment in trying to find an answer to this last question. Radiographs of one each of the encased nickel clad and the partly monel sleeved round steel dowels were taken before they were placed in the tidal water. Periodic radiographs will have to be taken hereafter.

CONCLUDING COMMENTS

While the effect of dowel corrosion as, at least, a contributory cause of malfunction of expansion joints had been established satisfactorily, the magnitude of this effect in contraction joints is still somewhat in doubt. It is hoped that the accelerated test will allow evaluation of the various protective applications reasonably soon. As to their performance in the pavement, an answer may not be as near at hand.

The selection of the Connecticut Turnpike for the experimental installations was a good one, as far as heavy traffic and load applications are concerned. It proved to be a bad one, however, as concerns any attempts to remove dowel blocks from the pavement. Even the taking of measurements at joints is disturbing to the State Police.

It is to be hoped that research by others will help to create a better understanding of the dowel corrosion problem, and that definite standards for corrosion resistant applications, of whatever nature, may ultimately derive therefrom.

ACKNOWLEDGMENTS

The author wishes to acknowledge the assistance given in the preparation of this paper by F. Sternberg and O. A. Strassenmeyer of the Division of Research and Development. Gratitude is also expressed to the various cooperating manufacturing concerns.

Investigational Concrete Pavement in Minnesota: 18-Year Report

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This 18-year report concludes the 1940 investigational concrete pavement project in Minnesota. The project was one of six constructed by various states in cooperation with the Bureau of Public Roads for the purpose of evaluating certain principles of concrete pavement design and the relative performance of such pavements over a period of years. Particular emphasis was placed on the effects of joint spacing, reinforcement, and pavement cross-section.

This report summarizes much of the earlier reported data pertaining to design and construction features, climate, traffic, joint movements, shrinkage, compressive stresses, and joint materials. In addition, the relative performance of the pavement designs are compared on the basis of the 1950 and 1958 condition surveys. Such physical characteristics as roughness, transverse and longitudinal cracking, faulted joints, corner cracks and blowups are related to the design features of the project.

Conclusions are presented on a considerable number of the experimental features of the project.

●THE MINNESOTA experimental concrete pavement was constructed in 1940 as a part of a cooperative joint spacing investigation sponsored by the Bureau of Public Roads. Five other similar experimental pavements were built in California, Kentucky, Michigan, Missouri and Oregon. The purpose of these experimental projects was to study and evaluate various fundamental principles of concrete pavement design and the relative performance of such pavements over a period of years. Two of the main objectives were:

1. To determine whether expansion joints could be eliminated or spaced at long intervals in plain concrete pavements with closely spaced contraction joints.
2. To compare the performance of reinforced concrete pavement with that of plain concrete pavement.

Data from the Minnesota project have been reported at various times during the life of the pavement. The general layout and special design features were described by Lang (1). The results of observations and measurements taken through July 1944 were reported by Lang (2). A ten-year report, including data through 1950 was reported by Carsberg and Velz (3).

This final 18-year report includes a summary of the previously reported information and conclusions without repetition of all the basic data. In addition, the results of a visual condition survey made in late August 1958 are included as new information, along with roughness and traffic data.

In the eight years after 1950, the pavement underwent considerable change.

Cracking became prevalent, joints and cracks were filled with rubber-asphalt joint sealing compound, and many of the spalled joints were patched with bituminous mixture. It was not possible to measure joint movements nor was it possible to make extensive, accurate measurements of joint faulting. Although some further information might be obtained on the condition of the pavement, it is improbable that this information could be analyzed relative to the experimental features of the project with a desirable degree of reliability.

DESIGN FEATURES

The experimental project, consisting of 8.1 mi of 22-ft concrete pavement, is located in southwestern Minnesota on State Highway 60 between Worthington and Brewster. This location was selected because: (a) the grade line was comparatively flat (maximum 2 percent); (b) the subgrade was a reasonably uniform A-7-6 clay loam soil; (c) the country was open, thus eliminating the influence of trees, buildings and snow accumulations; (d) it was situated in an area where marked variations in temperature and moisture occur; and (e) its length satisfied the requirements of the program.

As shown in Figure 1, the project was divided into 17 divisions numbered consecutively starting at Worthington. They do not correspond numerically with the section designations given in the Bureau of Public Roads program. However, as shown in the upper diagram, Divisions 3 to 15 inclusive are identical with the program, except that more than one panel length was used. Panel lengths of 15, 20, 25, 30 and 60 ft were used in the various divisions as indicated in the figure. Expansion joints were generally spaced at intervals of approximately 120, 400, 800 and 5,260 ft.

In addition to these design features, there were some 30 different expansion joint designs and 18 different contraction joint designs on the project. These variables consisted of the use of four types of copper seals, three expansion joint filler materials, four top sealing materials, and doweled and undoweled joints. These designs are discussed later in connection with joint performance.

MINN DIVISION NO	1		2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
BPR SECTION NO	NOT IN BPR PROGRAM			7	6	5	4	3	2	1	2	3	4	5	6	7	NOT IN BPR PROGRAM	
LENGTH IN MILES	1 09		0 27	0 27	0 25	0 27	0 27	0 62	0 61	1 00	0 61	0 68	0 27	0 27	0 25	0 27	0 27	0 79
PAVEMENT SECTION	9 6 9		7 7 7	7 7 7	9 6 9	9 6 9	9 6 9	9-6 9	9 6 9	9-6-9							7 7-7	9 6-9
EXPANSION JOINTS: JOINTS	YES		YES	YES	YES	YES	YES	YES	YES	NO	SAME AS MINN DIVISION NO 9	SAME AS MINN DIVISION NO 7	SAME AS MINN DIVISION NO 6	SAME AS MINN. DIVISION NO 5	SAME AS MINN DIVISION NO 4	SAME AS MINN DIVISION NO 3	YES	YES
DOWELS	YES		YES	NO	YES	YES	YES	YE'	YES								YES	YES
SPACING INTERVAL IN FEET	160 240 320' 480	640 905 1245 1740	120 125'	120 125	120	120 125'	120 125	400 420	795' 800' 810								120' 125	120' 200 240' 480' 1020' 1050'
CONTRACTION JOINTS: DOWELS	YES	NO	YES	NO	YES	YES	NO	NO	NO	NO							YES	YES
NUMBER OF 15 PANELS		128	24	24		24	24	56	53	88							24	
NUMBER OF 20' PANELS	96		18	18		18	18	40	40	66							18	
NUMBER OF 25 PANELS			15	15		15	15	32	32	52							15	
NUMBER OF 30 PANELS	63		12	12		12	12	28	27	44							12	125
NUMBER OF 40' PANELS																		5
NUMBER OF 60' PANELS					22													4
REINFORCEMENT	NO		NO	NO	WIRE MESH	NO	NO	NO	NO	NO							NO	SPECIAL BAR

Figure 1. Layout of the project.

CONSTRUCTION FEATURES

Subgrade

Paving operations started on August 6, 1940 and ended on September 20, 1940. The pavement was laid directly on the A-7-6 soil subgrade, except that subgrade paper was used on the northeastern half of the project. The soil density in the upper 18 in. of the roadbed averaged 96 percent as compared with standard laboratory moisture-density tests. However, there was considerable variation in the density with a maximum of 122 percent and a minimum of 71 percent.

Subgrade performance has been reasonably satisfactory throughout the project, except for some differential frost heaving at transitions from cut to fill sections. Although high joints have frequently occurred elsewhere on this type of soil, their occurrence on this project has been moderate, the 60-ft panels being the only ones to develop objectionably high joints. Pumping has not been noted on the project, although deflection at a considerable number of joints has been observed at various times. Some faulting has occurred in the later years.

Concrete

The aggregates were washed sand and gravel from a deposit located about 33 mi west of the project. The cement was standard Type 1 portland cement of one brand. These materials were batched from a trackside proportioning plant and produced concrete having the physical properties given in Table 1.

TABLE 1

PHYSICAL PROPERTIES OF CONCRETE

Flexural Strength		
6- by 6- by 36-in. beams	7 days	563 psi
6- by 6- by 36-in. beams	14 days	642 psi
Compressive Strength		
6-in. modified cubes	28 days	4,536 psi
6-in. diameter cores	150 days	5,451 psi
Thermal Coefficient of Expansion		
Lab.: 80-120 F	148 days	0.00000545
Field: 73-105 F	4 years	0.00000530
72-95 F	5 years	0.00000499
75-99 F	8 years	0.00000457
Modulus of Elasticity		4,300,000
Consistency—Slump Cone Method		½ to 1½ in.
Water-Cement Ratio		5.81 to 6.11 gal. per sack

The concrete was placed by vibratory equipment of the tubular, internal type supplemented by a portable, high-frequency vibrator used along with joints and forms. Immediately after brooming, the pavement was covered with impermeable, fiber-filled paper and cured for at least 72 hours.

CLIMATOLOGICAL DATA

The monthly maximum, minimum and mean air temperatures from August 1940 through July 1944 are shown in Figure 2. The mean monthly air temperatures ranged

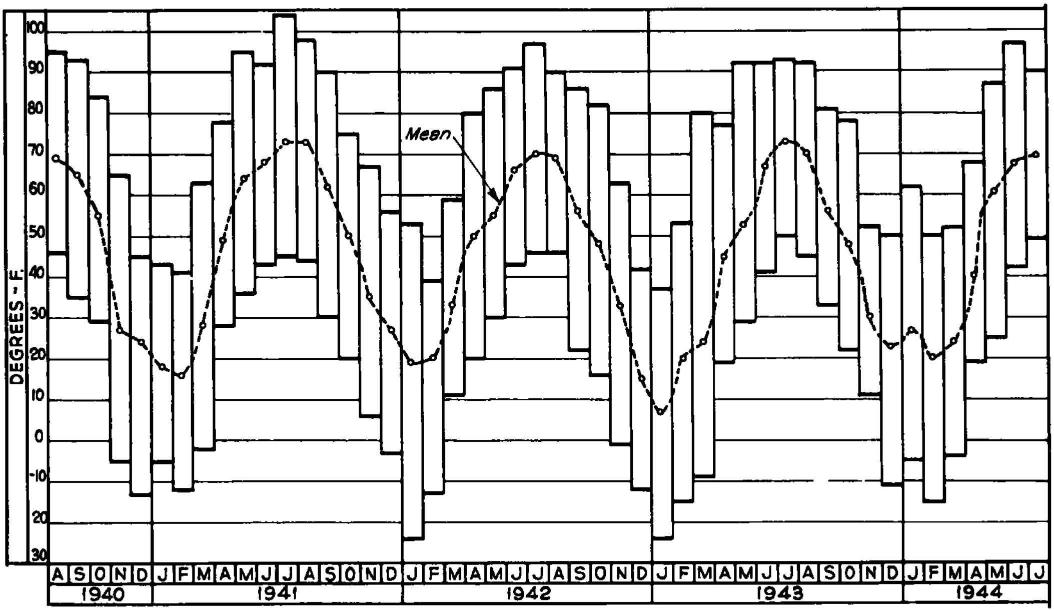


Figure 2. Air temperature data.

from maximums of 70 to 73 F in July and August to minimums of 7 to 20 F in January and February. The highest recorded air temperature was 104 F in July 1941 and the lowest was -24 F in January 1942 and 1943.

The monthly precipitation for the same period is shown in Figure 3. The highest monthly precipitation occurred in July 1943, amounting to slightly over 9 in. The total annual precipitation ranged from 28 to 33 in., occurring mostly in the spring and summer in the form of rain.

These data may be considered typical for the project area for the years subsequent to 1944.

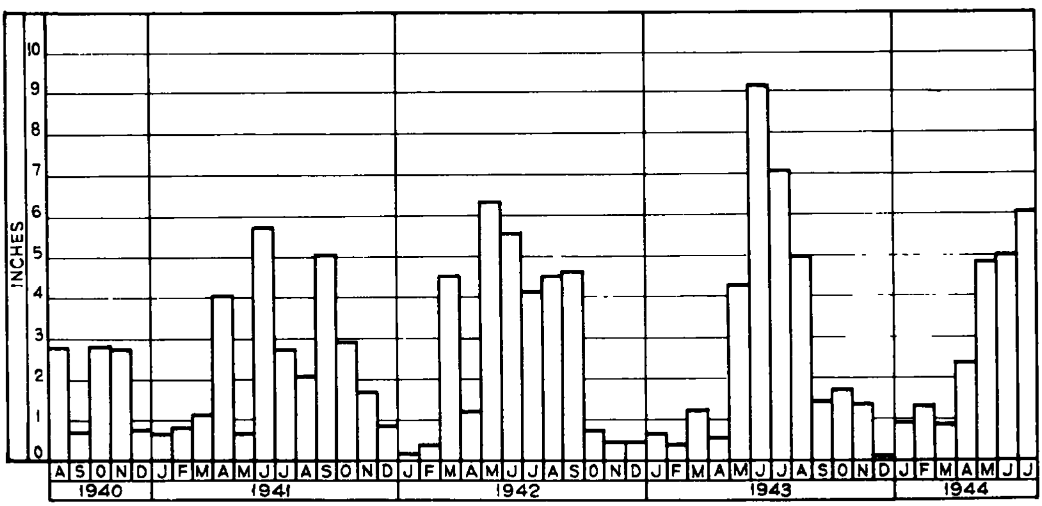


Figure 3. Precipitation data.

TRAFFIC

The amount and character of traffic on this project has changed considerably during the past 18 years. In 1941, the year after construction, the average daily traffic (ADT) was 572 vehicles including 88 trucks. Traffic increased only slightly during the war years and through 1947. Thereafter, traffic increased each year, reaching a peak ADT of 2,386 in 1955. Commercial vehicles showed a similar increase and leveled off at about 400 per day after 1953. The average daily traffic for each year since 1941 is shown in Figure 4.

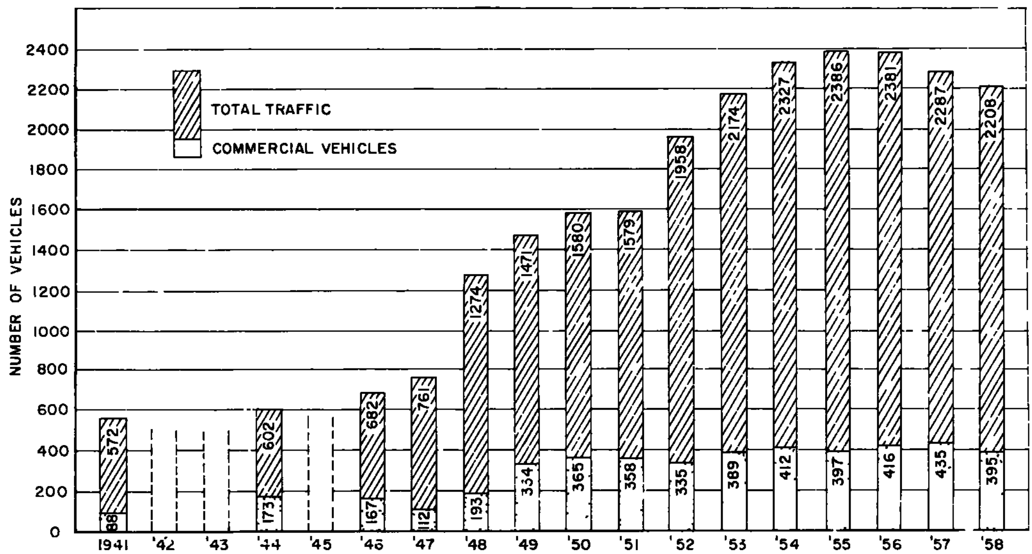


Figure 4. Average daily traffic.

The character of the traffic for various years from 1936 to 1958 is given in Table 2. The increase in multi-axled semitrailers was particularly noticeable, increasing from a total of 2 in 1941 to 107 in 1950 and 148 in 1958. This increase in larger trucks was responsible to a large degree for the increase in pavement distress after 1950.

JOINT MOVEMENTS

Daily, annual and permanent changes in joint openings were determined at over 700 joints on this project at various times between October 1940 and August 1948. These data were presented in detail in the ten-year report, so only a brief summation is included in this report.

TABLE 2

24-HOUR AVERAGE ANNUAL DAILY TRAFFIC

Vehicle Type	1936 1937	1941	1944	1946	1948	1950	1952	1958
Passenger cars	280	484	429	515	1,081	1,215	1,623	1,813
Single unit trucks								
2-axled	39	73	123	124	142	246	214	194
3-axled	3	11	37	2	2	5	10	9
Tractor–semitrailers								
3-axled	-	2	10	29	25	55	41	48
4-axled	-	-	1	8	18	45	55	65
5-axled	-	-	-	1	2	7	4	35
Trucks with trailers	-	-	-			2	4	35
Busses	2	2	2	3	4	5	7	9
Total vehicles	324	572	602	682	1,274	1,580	1,958	2,208

Daily Changes in Joint Openings

The width of any individual joint opening is continually changing. It is affected not only by the daily temperature changes, but also by seasonal changes in temperature and moisture, the length of adjoining panels, the expansion joint interval, the location of the joint (contraction) in relation to the nearest expansion joint, prior joint movements, and the age of the pavement. Thus, the daily joint movements were in a forever changing pattern. However, based on the readings obtained on July 24, 1944 when the temperature ranged from 73 to 106 F, certain observations were made which are believed to illustrate the daily movements during the summer. Table 3 gives the average change in joint openings on this day for various combinations of panel length and expansion joint interval.

TABLE 3

DAILY CHANGE IN JOINT OPENINGS, JULY 24, 1944, TEMPERATURE RANGE 73-106 F

Expansion Joint Interval	Contraction Joint Interval	Average Daily Change in Joint Opening (in.)	
		Contraction	Expansion
120	30	0.067	0.068
120	60[1]	0.075	0.171
125	25	0.060	0.023
420	15	0.034	0.065
420	30	0.058	0.055
795	15	0.032	0.016
810	30	0.034	0.020
5,260	15	0.030	0.025
5,260	30	0.048	0.025

[1] Reinforced panels.

The greatest joint movement was recorded in the 60-ft reinforced panels, where every other joint was an expansion joint. These panels showed restraint or closing of the contraction joint at approximately one-half the daily rise in temperature or at about 90 F. Above this temperature, the entire 120 ft continued to expand as a unit, thus accounting for the large movements at the expansion joints.

Restraint also occurred at approximately 90 F in the 15- and 30-ft panels where expansion joint intervals were 400 ft or more. This restraint was more pronounced through the central portions of the sections between expansion joints. Contraction joints in the vicinity of an expansion joint tended to develop relatively large movements. This was true in the cases of annual and permanent movements as well as the daily movements.

In general, the daily change in joint openings decreased as the expansion joint interval increased and the panel length decreased. Therefore, short panels and wide spacing or omission of expansion joints would appear to be beneficial in reducing infiltration of water and dirt through or into the joints and also in providing maximum load transfer by aggregate interlock across contraction joints.

Annual and Permanent Changes in Joint Openings

Of the 714 joints originally provided for measuring these movements, only 438 joints could be considered in 1948 because of the desire to eliminate the influence of cracked panels. On this basis, it was possible to obtain data on all combinations of panel lengths and expansion joint intervals except the 125-ft expansion 25-ft contraction sections.

Some of the interesting points relating to these measurements are:

1. The general tendency for the contraction joint next to an expansion joint to open considerably more than other contraction joints in the section. However, the general trend indicates an increase in opening as the joint location approaches the expansion joint.

2. The erratic behavior of individual contraction joints during any given season or year or from year to year.

3. The proportionately greater closure of the expansion joints during the first expansion cycle after construction as compared with closures during later cycles.

4. The small movement of the expansion joints after 8 yr where the expansion joint interval was 400 ft or more in length; as compared to the larger movement of the expansion joints where the joint intervals were on the order of 120 ft; and expecially as compared to the 60-ft reinforced panels with alternating expansion and contraction joints where expansion joint movements were undiminished after 8 yr.

5. The general trend for the accumulated opening of the contraction joints per expansion interval to very nearly equal the accumulated closure of the expansion joint when sufficient expansion space was available, such as in the 120-ft expansion intervals.

6. The increasing rate of closure of expansion joints as the interval between these joints was lengthened. At the end of 8 yr, the expansion joints (originally 1 in.) at the ends of Div. 9 (1 mi without expansion joints) were considered closed; expansion joints at 400- and 800-ft intervals were closed 85 to 90 percent of their original 1-in. width; whereas the expansion joints at 120-ft intervals were only closed about 57 percent.

7. The advantage of short panels and long spacing or omission of expansion joints in controlling joint movements. Figure 5 shows a summary of average contraction joint openings in the winter and summer of 1948 for all combinations of contraction and expansion joint spacings. The smaller summer openings of contraction joints are associated with the longer expansion intervals, and the smaller winter openings are associated with the shorter panel lengths.

Seasonal Moisture Change and Shrinkage

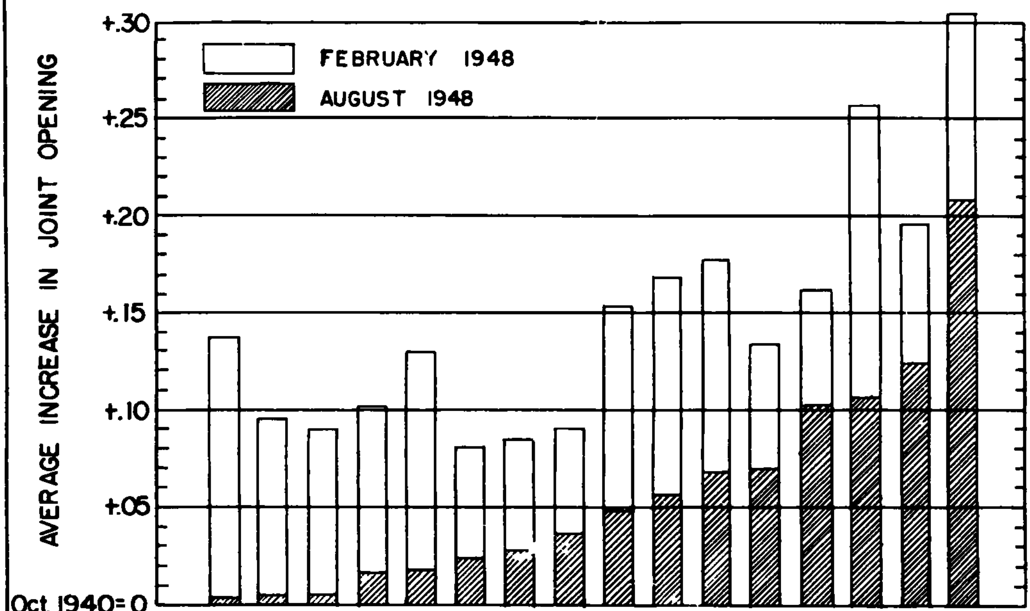
Seasonal moisture changes and shrinkage of the concrete tend to add additional expansion space in the pavement. On this project, on a 120-ft expansion 15-ft contraction section, the computed change in length due to these factors was determined to be a shortening of 0.3620 in. by July 1944, and 0.4734 in. by August 1948.

These increases in expansion space due to shrinkage and the compensatory effect of seasonal moisture change are both in addition to the original joint space build into the pavement. Neglecting any reduction in this space due to foreign material infiltrating the joint openings, this would theoretically mean that the original expansion joints would not be required to function as such at temperatures below 95.1 F after 4 yr or below 107.8 F after 8 yr of service.

That infiltrated material may become a serious matter, where the pavement design places no restraint on the expansion and contraction of individual panels, is apparent from inspection of Figure 6. This figure shows the progressive change in opening of both the expansion and contraction joints on this particular section. As built, all the expansion space (1 in.) was concentrated in the expansion joint inasmuch as there were no openings at the dummy-type contraction joints. The change in distribution of this space and the consequent migration of the panels toward the expansion joints are clearly shown. By July 23, 1944 the space in the expansion joint was reduced from 1 in. to 0.5317 in. and by August 18, 1948 to 0.2499 in. Meanwhile, the contraction joints had accumulated a total opening of 0.5783 in. by July 1944 and 0.8961 in. by August 1948. It is interesting to note that the sums of the openings on these dates were 1.110 in. and 1.146 in. respectively, 0.110 and 0.146 in. greater than the original expansion space built into the section.

There were seven contraction joints in this section. Hence, the average contraction joint opening on August 1948 was 0.1280 in. and in February 1948 the average opening was 0.1936 in. This illustrates the undesirable result of incorporating excessive expansion space in the pavement. Such openings facilitate the entrance of foreign material into the joints, decrease the possibility of load transfer across the joints by

CONT. JOINT INTERVAL	30	20	25	20	25	15	15	15	20	30	30	20	15	30	25	60
EXP. JOINT INTERVAL	5260	800	800	5260	5260	5260	795	420	120	420	810	400	120	120	400	120



AVERAGE CHANGE IN JOINT OPENINGS (Inches)																
AUG. 1948	0.004	0.005	0.005	0.016	0.017	0.025	0.029	0.038	0.049	0.058	0.068	0.070	0.102	0.108	0.124	0.208
FEB. 1948	0.139	0.096	0.091	0.101	0.129	0.081	0.085	0.091	0.154	0.170	0.178	0.135	0.161	0.258	0.196	0.309

Figure 5. Average increase in contraction joint openings.

interlock of the slab edges, and, in effect, create free edges in the pavement.

Longitudinal Compressive Stresses

The longitudinal compressive stresses due to thermal expansion in restrained concrete were determined at the center and at the ¼-points of Division 9 (1 mi without expansion joints) for a temperature of 112 F. The instrumentation, measurements and calculations for these determinations were included in the 10-yr report. The resultant unit stresses and thermal coefficients were as follows:

	Unit Compressive Stress, psi			
	Center Point	East ¼ Point	West ¼ Point	Thermal Coefficient
August 24, 1944	679	679	671	0.00000530
August 2 and 3, 1945	662	692	722	0.00000499
August 17 and 18, 1948	602	602	654	0.00000457
Lab. determination (shortly after construction)				0.00000545

There appears to be a gradual reduction in the thermal coefficient as indicated by the field determinations.

The unit stresses in the restrained concrete were quite low, being only 1/7 the ultimate compressive stress of the concrete. Thus, it is indicated that no serious compressive stresses have developed in this pavement up to 8 yr after construction. How-

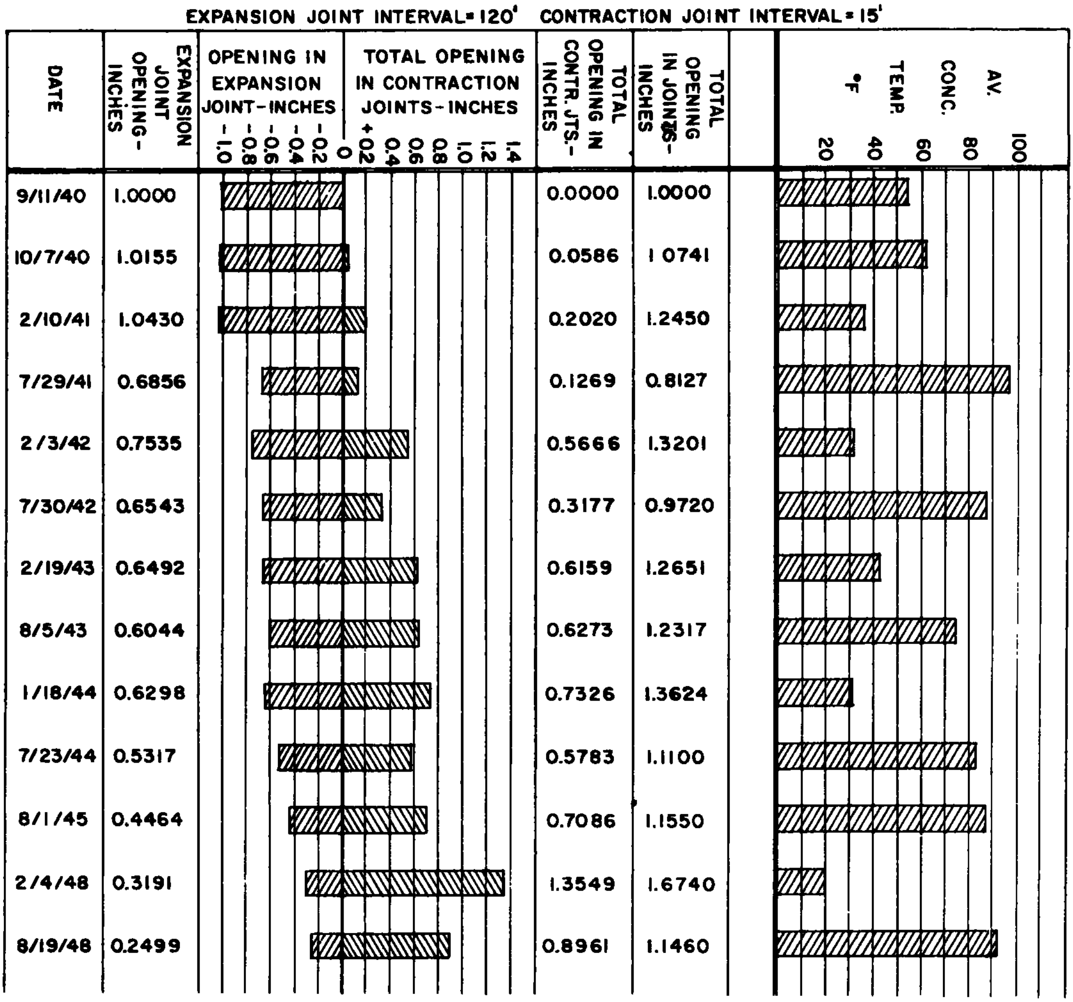


Figure 6. Progressive closure of expansion joints and cumulative opening of contraction joints.

ever, the possibility of these compressive forces being concentrated should not be overlooked, especially when associated with vertical deflection of the joints caused by traffic.

JOINT DESIGNS AND MATERIALS

Expansion Joints

The expansion joints on this project were all 1 in. in width, and the intervals between these joints ranged from 120 ft to 1 mi. Variables in design included: the use of copper seals in some joints, whereas in others this seal was omitted; three different filler or core materials including premolded cane and premolded wood fiber and a poured type consisting of ground cork and asphalt; four top sealing materials including asphalt-diatomaceous earth mixture, a latex-oil mixture, a manufactured rubber material (Rubber Associates) and premolded rubber strips manufactured by the Goodrich Rubber Company. In addition to these variables, some joints did not include dowels or other load transfer devices. The various combinations resulted in a total of 30 different expansion joint designs. A typical expansion joint is shown in Figure 7 (a).

The copper seals used on this project brought about little if any reduction in the seasonal vertical movement of the expansion joints relative to the center of the adjacent panels. There was an indication that, in general and where panel lengths are not excessively long, copper seals may for a time be somewhat beneficial in reducing the magnitude of seasonal deformations during the early life of the pavement.

Based on data obtained up to 1944 on the effectiveness of top-sealing materials, it was concluded that there was little difference among asphalt-diatomaceous earth, latex-oil and premolded rubber strips. All of these indicated several times as much vertical movement of the joints as the Rubber Associates material.

Examination of the joints in 1950 showed that the closure of the joints had resulted in the general extrusion of the filler and top-sealing materials. The need for non-extrusive joint fillers was obvious, expecially where there were long intervals between the expansion joints.

Contraction Joints

Dummy type contraction joints were used exclusively on this project. A total of 18 different designs were used which involved various metal seals, the use of asphaltic, latex-oil and rubber top seals, and the use of dowels in some cases and their omission in others. Various designs are shown in Figure 7.

Based on seasonal vertical movements measured in 1944 and 1948, Rubber Associates material was the only one to show a positive reduction in joint deformation. However, in 1950, a survey of 31 contraction joints sealed with Rubber Associates material showed that only 7 were not open and rated as being in fair to good condition. The other 24 joints were open from $^1/_{16}$ to $^1/_4$ in. and generally in poor condition. These joints were in various divisions of the project and associated with various panel lengths. Maintenance of the joints was not permitted except where absolutely necessary, throughout this 10-yr period.

The effect of contraction joint design upon spalling of the concrete adjacent to the joint is discussed later in this report in connection with the condition of the pavement in 1958. However, it should be pointed out here, that the joints with rubber types of top-sealing materials (latex-oil and Rubber Associates) had significantly less spalling than the joints sealed with asphalt-diatomaceous earth mixture.

GENERAL PROJECT CONDITION

From the time of construction until 1950, the project was considered to be generally in good condition except for certain localized cracking. There were several areas where considerable diagonal cracking had occurred as a result of frost heaves in cut-fill locations. Some longitudinal cracking had occurred near the quarter points because of variable subgrade support under the pavement slab. A relatively small amount of

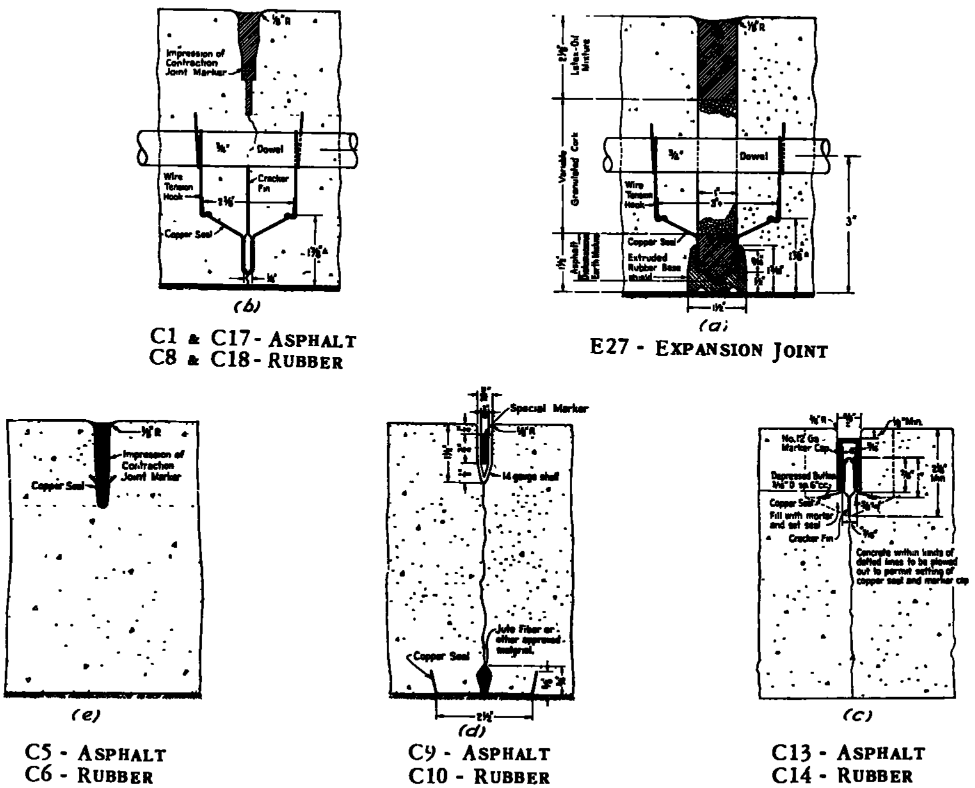


Figure 7. Typical joint designs.

transverse cracking had occurred, some of which was caused by temperature change and frozen subgrade.

By 1952, there was a distinct difference in the appearance of the concrete around some of the transverse joints. Investigation showed that many fine cracks, of the type referred to as "D" cracks, had occurred in the vicinity of the joints. D cracks appear as fine, raised lines composed of soluble materials leached from the cracks and deposited on the surface of the concrete. In some instances, these cracks appeared only at the intersection of the transverse and longitudinal joints. At other places they occurred across the entire pavement and parallel to the transverse joint. By 1958, this cracking had progressed to the point where the concrete was disintegrated at approximately 15 percent of the transverse joints to the extent that patching with bituminous material was required. No such cracking occurred at the warping joints placed in the 30-ft reinforced panels in Division 17 for the purpose of relieving stress caused by warp or curl. These joints were not included in the general joint analysis.

The pavement surface has shown no sign of scale; although, at heavily traveled side roads there are signs of abrasive wear caused by gravel tracked onto the pavement.

The concrete in the center portion of the pavement panels, along the center joint and along the edges (except near transverse joints) appears to be generally sound. Cores taken in 1940 and tested at 150 days had an average strength of 5,451 psi (Table 1). Twelve cores were drilled from the sound concrete in 1959 for strength determinations. They were distributed throughout the project, and were taken within 8 ft from a core drilled in 1940. The comparison of the core strengths (Table 4) indicates that in every case the concrete was stronger after 19 yr than it was at 150 days, the 1959 average strength being 7,660 psi.

TABLE 4

COMPARISON OF CONCRETE COMPRESSIVE STRENGTHS WITH AGE

Division No.	Compressive Strength (psi) 150 Days	19 Years	Increase
1	5,632	9,191	3,559
2	6,184	8,164	1,980
3	6,500	9,627	3,127
5	5,673	6,344	671
6	6,165	9,505	3,340
9	5,997	7,203	1,206
11	4,421	8,235	3,814
11	4,338	6,912	2,574
12	4,208	5,127	919
13	5,614	7,354	1,740
13	6,115	8,136	2,021
15	5,156	6,124	968
Average	5,500	7,660	2,160

Roughness

The roughness of the pavement has increased considerably during the last 8 yr. This has been caused primarily by the failure of the transverse joints and the patching that has been required. Because of the roughness, trucks were sometimes driven with the right wheels on the shoulder.

The roughness of the pavement was again measured in 1958 by a roughness recorder, which is a duplicate of the machine described by Buchanan and Cotudal (4). The average roughness obtained at various times and the change in roughness are as follows:

Date	In. per Mi	Percent Change from Nov. 1941
November 1941	85	0
February 1942	84	-1
July 1944	96	+13
November 1949	100	+18
November 1950	105.5	+24
September 1958	124	+46

These values, while expressed in inches per mile, should not be construed as being absolute values in these terms; they represent the accumulation, in inches, of spring deflection of the machine as influenced by the pavement roughness. The values are significant only for comparative purposes, showing that there has been a steady, progressive increase in roughness.

The relationship of roughness to panel length and cracking is given in Table 5. It is significant to note that the roughness index of 104 for the 60-ft reinforced panels was much lower than the values for other panel lengths, which ranged from 119 to 126 in. per mile. The 60-ft panels also had the lowest increase in roughness (9 in.) from the previous roughness survey in 1950. Other increases in roughness ranged from 15 to 28 in., the latter being for the 30-ft reinforced panels. The increased roughness of the 30-ft reinforced panels was caused by the extreme disintegration of the concrete at the transverse joints and the required patching.

Transverse Cracking

The relationship between percentage of transversely cracked panels and panel length for non-reinforced pavement was about the same in 1958 as in 1944, but the amount of cracking had increased about ten times. In both years the percentage of cracked

TABLE 5

1950 AND 1958 CRACK AND ROUGHOMETER SURVEYS: TRANSVERSE CRACKING AND ROUGHNESS INDEX AS RELATED TO PANEL LENGTH (ALL DIVISIONS INCLUDED)

Panel Length	Percent of Total Panels Cracked Transversely		Average Spacing of Transverse Opening		Number Transversely Cracked Panels Per Mile		Av. No. Transverse Openings Per Mile		Roughness Index (in. per mi)	
(ft)	1950	1958	1950	1958	1950	1958	1950	1958	1950	1958
15	1.2	4.5	14.8	14.4	4	15	357	368	107	126
20	3.3	6.7	19.4	18.7	9	18	272	281	104	119
25	13.2	23.7	22.1	20.2	28	50	239	261	104	123
30	22.1	39.1	24.6	21.6	39	68	215	244	105	122
30 reinforced	28.6[1]	40.5	23.3[1]	21.4	50[1]	70	226[1]	246	92	120
30 reinforced and with cracker strip	0	2.4	15.0	14.8	0	4	352	356	104	125
60 reinforced	17.7	43.8	51.0	41.7	16	38	104	126	95	104

[1]Value in 1950 report was in error.

30-ft panels was about eight times that of the cracked 15-ft panels. Figure 8 shows the percentage of transversely cracked panels by panel length for the years 1944, 1950 and 1958. The percentage of cracked panels in 1958 varied from a low of 4.5 for the 15-ft panels to a high of 39.1 for the 30-ft panels. In general, the percentage of cracking increases approximately in relation to the square of the panel length in excess of 10 ft.

Table 5 gives the crack frequency for the various panel lengths as obtained from the 1950 and 1958 crack surveys. It is apparent that the 15- and 20-ft panels have been fairly effective in controlling transverse cracking. In 1958, less than 7 percent of

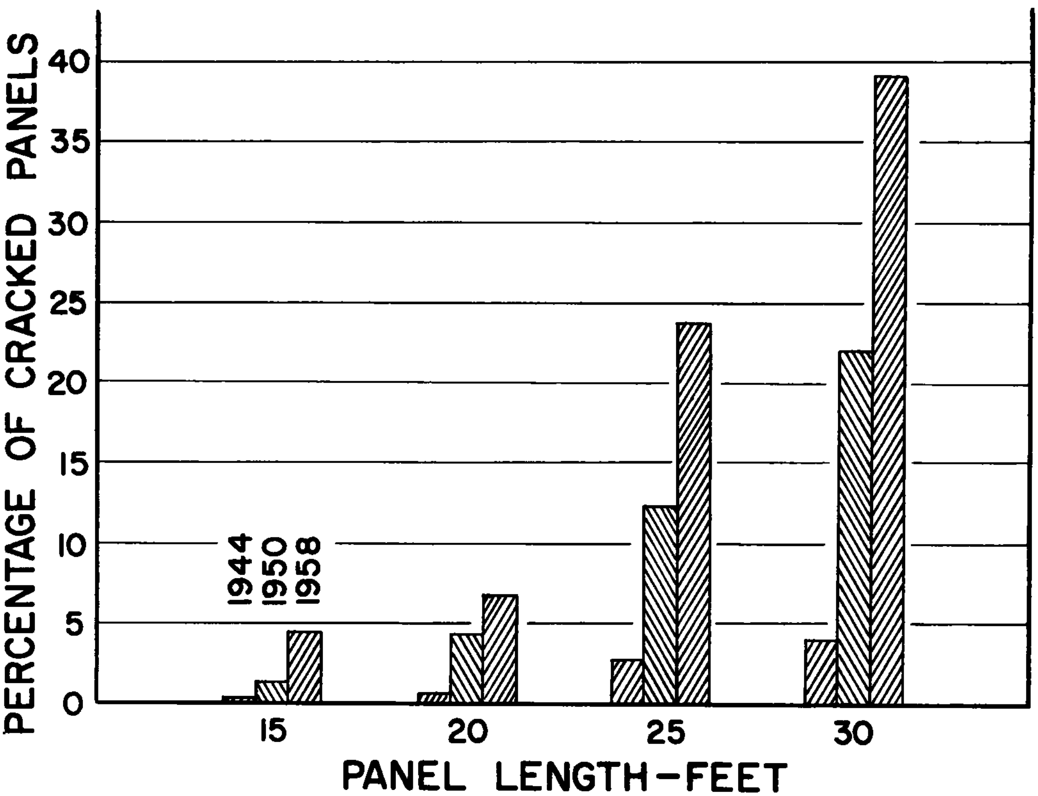


Figure 8. Relationship of transverse cracking to panel length.

TABLE 6

1950 AND 1958 CRACK SURVEYS: CRACKING AS RELATED TO PAVEMENT SECTION AND EXPANSION INTERVAL FOR VARIOUS PANEL LENGTHS (NON-REINFORCED PAVEMENT)

		Non-Restrained Concrete Expansion Joint Interval (120 and 125 ft)				Restrained Concrete Expansion Joint Interval (400, 800 and 5,260 ft)			
		No. of Transverse Cracks Per Mile		Feet Longitudinal Cracks Per Mile		No. of Transverse Cracks Per Mile		Feet Longitudinal Cracks Per Mile	
Panel Length (ft)	Section	1950	1958	1950	1958	1950	1958	1950	1958
15	7 in.	9.2	33	110	469				
15	9-6-9	0	15	323	975	4	12	140	651
20	7 in.	5.5	29	84	310				
20	9-6-9	1.8	5	125	777	11	18	613	1,354
25	7 in.	1.8	26	49	401				
25	9-6-9	30	72	63	736	36	51	458	991
30	7 in.	40	95	0	260				
30	9-6-9	42	92	546	2,057	38	58	676	1,292
Average	7 in.			61	360				
Average	9-6-9			262	1,136			472	1,072

these panels were cracked as compared with 24 percent or more for the longer panels.

Table 6 gives the frequency at which transverse cracking occurred in the non-reinforced concrete in both restrained and non-restrained sections. The crack frequency was greater in the 7-in. uniform depth section for 15- and 20-ft panels than in the 9-6-9 section. Theoretically there should be less cracking in the 7-in. section because of the greater depth. Crack frequency was about the same for both pavement sections for the 30-ft panels. The restrained sections (all 9-6-9) show increased crack frequency with increase in panel length.

Figure 9 shows a transverse crack open approximately $^3/_{16}$ in. Cracks such as this are sealed periodically during routine joint maintenance.

Longitudinal Cracking

Longitudinal cracking has increased at a higher rate than transverse cracking. In 1944, there were 405 ft of longitudinal cracks on the project. In 1950, longitudinal cracks had increased to 2,685 ft, and, by 1958, there were 7,375 ft of cracks.

Table 6 gives the comparison between the longitudinal cracking in the restrained and non-restrained concrete. Longitudinal cracking increased about 125 percent from 1950 to 1958 in the restrained concrete, whereas it increased from 300 to 500 percent in the non-restrained sections. The over-all average cracking for the restrained and non-restrained concrete in

Figure 9. Typical transverse crack in non-reinforced pavement. Crack open 3/16 in. in 30-ft panel. Division 1, Sta. 220 + 93.

1958 was about the same for each, being 1,072 and 1,136 ft per mile, respectively.

Table 7 gives the distribution of the longitudinal cracking in relation to the panel length, expressed in terms of percent of panels cracked, feet of crack per panel and feet of crack per mile. This table indicates that, in all instances, there was much less cracking in the 7-in. uniform section than in the 9-6-9 section where no reinforcement was used. For the 9-6-9 section the reinforced pavement had a little less cracking than the non-reinforced pavement. All the 30-ft reinforced panels contained longitudinal steel bars (½ in.) whereas only eight of the 96 panels 60-ft long contained longitudinal bars, the others being reinforced with 66-33 wire mesh (68 lb per 100 sq ft). These 60-ft reinforced panels had the least amount of longitudinal cracking, amounting to only 160 ft per mile.

Figure 10 illustrates the longitudinal cracking that has occurred on the project. Much of this continuous cracking reflects inadequate subgrade support for the 9-6-9 pavement section. There were 8 pairs of short restraint cracks in Division 9 (5,260-ft expansion interval) and 10 pairs of restraint cracks in Division 10 (800-ft expansion interval).

Faulted Joints

In 1958 the condition of the concrete around many of the joints was in such poor condition that the extent and magnitude of joint faulting could not be determined. Many of the joints that previously were shown as faulted were either severely cracked and spalled or patched.

In 1944 there were only five faulted joints whereas by 1950, this had increased to a total of 195. At both times the magnitude of the faulting did not exceed ¼ in. The data showed that: (a) dowels were effective in reducing faulting in all sections except the 60-ft reinforced panels; (b) in the 60-ft panels, 40 percent of the joints were faulted even though dowels were present; (c) more faulting was associated with the 7-in. uniform section than the 9-6-9 section; (d) faulting tended to increase as panel lengths increased; and (e) faulting tended to decrease as expansion intervals were increased.

Spalled and Patched Joints

Joint deterioration has taken place at a very rapid rate in the past 8 yr and has resulted in, what can be termed, complete failure of a great many joints. In 1950 there were 81 spalled joints and the spalls were of minor magnitude. In 1958 there were 897 spalled joints (48 percent of the total), at many of which the spalling extended as much as 2 ft from the joint. Of these, 390 were cracked and spalled so badly that they were patched with bituminous material.

Table 8 summarizes the data on the percentage of spalled joints and percentage of

TABLE 7

1950 AND 1958 CRACK SURVEYS: LONGITUDINAL CRACKING AS RELATED TO PANEL LENGTH AND PAVEMENT SECTION (ALL DIVISIONS INCLUDED)

Panel Length (ft)	Section	% of Panels Cracked Longitudinally		Ft of Crack Per Panel (Average)		Ft of Crack Per Mile (Average)	
		1950	1958	1950	1958	1950	1958
15	9-6-9	2.6	10.0	0.246	1.008	173	710
15	7 in.	1.0	5.7	0.156	0.667	110	469
20	9-6-9	8.7	20.1	0.992	2.365	524	1,249
20	7 in.	2.8	6.2	0.160	0.587	84	310
25	9-6-9	7.5	20.4	0.853	2.192	360	926
25	7 in.	0.8	5.8	0.117	0.950	49	401
30	9-6-9	15.0	27.2	1.854	4.177	653	1,470
30	7 in.	1.0	7.3	0	0.740	0	260
30 (reinforced)	9-6-9	4.8	13.5	0.817*	2.675	287*	941
30 (reinf. 15-ft cracker-strip)	9-6-9	4.0	9.7	0.308	2.290	108	807
60 (reinforced)	9-6-9	3.1	7.3	0.479	0.906	84	160

* Value in 1950 report was in error.

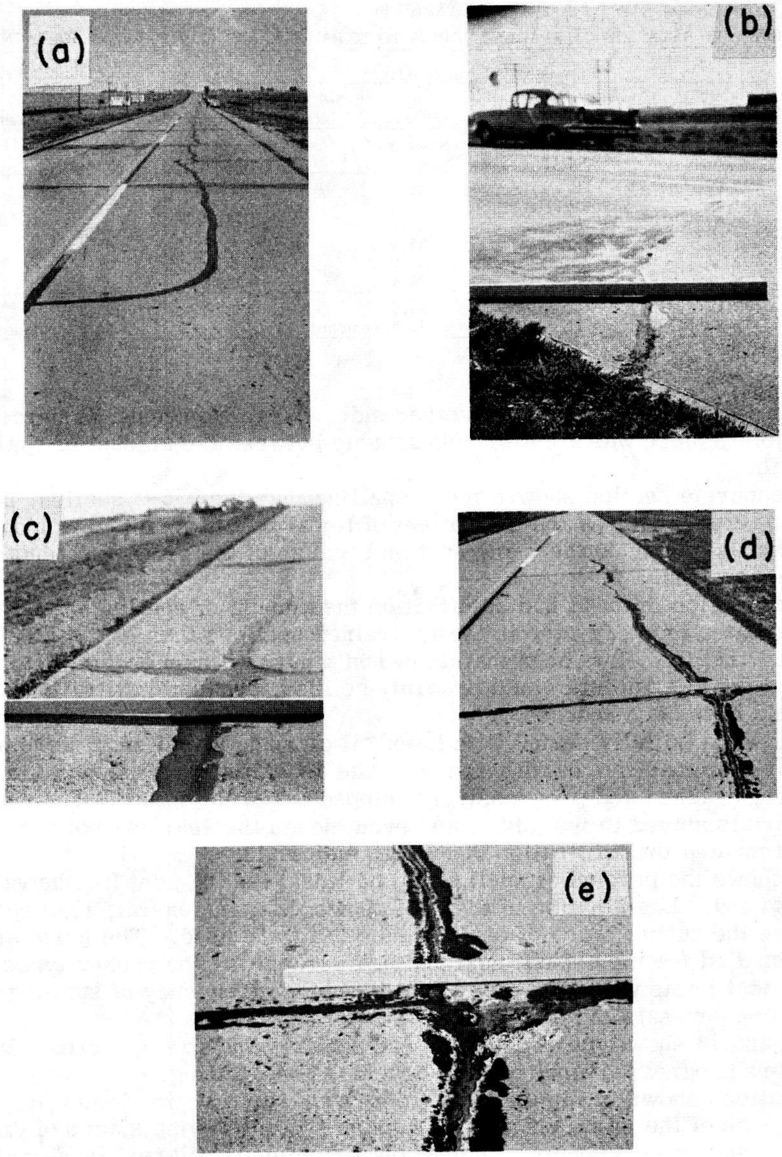


Figure 10. Typical longitudinal and diagonal cracking, as follows: (a) longitudinal crack, 30-ft panels, Div. 1, Sta. 221 + 75 to 223 + 50; (b) corner break, 25-ft panel, Div. 9, Sta. 415 + 95; (c) longitudinal crack, 30-ft panels, Div. 6, Sta. 333 + 10; (d) longitudinal crack, 30-ft panels, Div. 10, Sta. 480 + 70; and (e) close-up of crack at Sta. 480 + 70.

lineal feet of spalls for the different panel lengths, pavement sections, and expansion joint intervals. There was no consistent relationship between the percentage of spalled joints and the panel length in the 120-ft expansion interval. In the restrained sections (expansion interval 400 ft or more) the percent of spalled joints became larger as the panel length increased. It must be mentioned that joints were not always spalled the entire distance across the slab at the time of the 1958 survey. In some instances there was no spalling on one side of the

TABLE 8

1958 CRACK SURVEY: RELATION OF SPALLED JOINTS TO EXPANSION AND CONTRACTION JOINT INTERVAL

Contr. Joint Interval	Expansion Interval (120 ft) 7 in. Uniform Percent of Joints Spalled by Number of Joints	Expansion Interval (120 ft) 7 in. Uniform Percent of Joints Spalled by Length of Joints	Expansion Interval (120 ft) 9-6-9 Percent of Joints Spalled by Number of Joints	Expansion Interval (120 ft) 9-6-9 Percent of Joints Spalled by Length of Joints	Expansion Interval (400 ft +) 9-6-9 Percent of Joints Spalled by Number of Joints	Expansion Interval (400 ft +) 9-6-9 Percent of Joints Spalled by Length of Joints
15	37.5	8.1	28.1	5.5	34.2	5.7
20	31.9	11.1	34.7	7.4	52.5	15.8
25	43.3	9.0	25.0	5.9	59.3	19.0
30	47.9	18.2	35.4	9.7	69.1	23.2
30 ft reinf.			50.0	14.8	86.4	68.0
60 ft reinf.			36.4	7.3	60.0*	48.9*
Average	39.2 12.8% patched	10.8	31.6 11.3% patched	7.0	53.3 24.8% patched	18.4

*Expansion interval 240 ft.

center line but complete spalling on the other side. When expressed as percent of lineal feet of joint, there was a better relationship between the amount of spalling and the panel length.

The 7-in. concrete section showed more spalling than the 9-6-9 section, both when expressed as percent of joints and as percent of lineal feet of joint. It would seem that less spalling should occur on the 7-in. section because of the greater strength obtained with the thicker section.

Length of expansion interval had an effect on the amount of spalling. The amount of spalling was greater in every case in the restrained sections than in the 120-ft expansion intervals. Had the joints been maintained in a normal manner during the first ten years, the spalling at all joints would probably be less, because infiltration of foreign materials would have been reduced.

Some joints were initially sealed with latex-oil or rubber, but most were sealed with an asphalt-diatomaceous earth mixture. The asphaltic material became brittle in cold weather, cracked and progressively chipped out of the joints. The rubber and latex-oil materials stayed in the joints; and even though the seal was not perfect, either prevented or retarded the infiltration of foreign materials.

Figure 11 shows the percent of spall based on lineal feet of joint for the various types of joints used. Examination of the data shows that, in general, less spalling occurred where the rubber types of sealing material were used. The job average was 16.7 percent spalled for the asphalt seal and 5.6 percent for the rubber types. Figure 11 also shows that joints with copper seals had a higher frequency of spalling than the joints without copper seals.

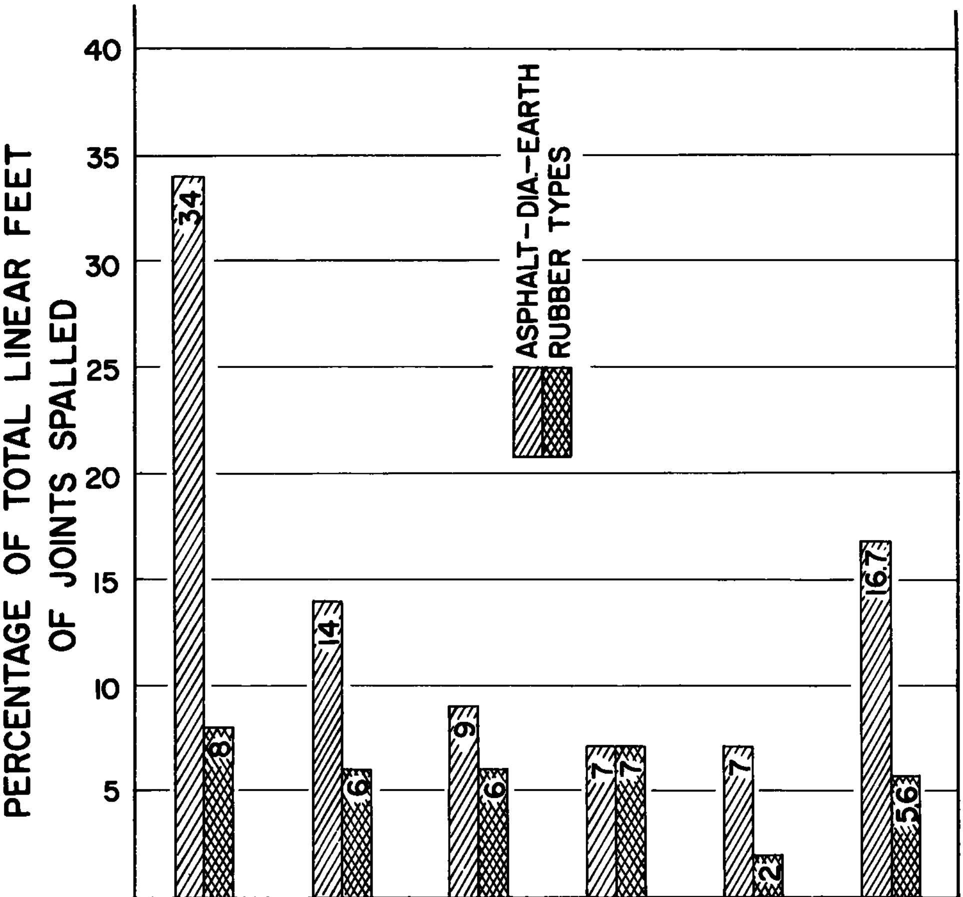
Figures 12 and 13 show typical cracking and spalling that has occurred. Illustrations are for the more progressed conditions of cracking and spalling.

The illustrations shown in Figure 12 are of doweled contraction joints in restrained concrete. At each of the joints shown, the concrete is in varying stages of disintegration, ranging from a relatively sound condition to complete failure. In Figure 12 (d), the left side of the joint shows the more progressed cracking whereas the lower right shows the D cracks that appear prior to the open fractures.

Figure 13 (a) and (c) show wider areas of distress at the edge of the pavement than the illustrations in Figure 12. These are undoweled joints in restrained concrete. The patched expansion joint shown in Figure 13 (b) had only one minor spall in 1950. Many contraction joints have the same type of crack and patch pattern.

Figure 13 (d) shows a series of patched joints. The joint in the foreground is a contraction joint. The others are successively a warping joint, in good condition, a patched expansion joint, followed by a series of patched contraction joints.

It is believed that much of the failure of the concrete at the joints was due to variable subgrade support caused by non-uniform subgrade density and leakage of water through the open joints and cracks. Although there was no pumping observed it was noted that blowing occurred in the vicinity of many joints. Blowing is associated with slab deflection and such deflections produce tension in the upper part of the slab and compression at a point immediately below the formed portion of the joint. These



DESIGN NO.	1 8 17 18	9 10	5 6 13 14	2 7 15 16	3 4 11 12	AVERAGE
COPPER SEALS	BOTTOM	BOTTOM	TOP	NO	NO	
DOWELS	YES	NO	NO	YES	NO	
TOTAL LINEAL FT. OF JOINTS	8580 506	14146 1012	6666 550	2244 550	3344 616	34980 3234

Figure 11. Relation of joint spalling to contraction joint design.

induced stresses could have caused fatigue failure of the concrete. Evidence of this was found in cores taken in D-crack areas near joints. Fracture planes in these cores were generally at about a 45-deg angle with the pavement surface.

Warping Joints

There are 61 warping joints on the project, and there was no spalling at any of them in 1958. However, 31 of these joints (about 50 percent) showed signs of D cracking at one or both ends. In no case did the D cracks extend more than 18 in. in from the edges of the pavement. In contrast to the relatively good condition of these warping joints, the adjacent contraction joints were badly disintegrated and patched.

Corner Cracks

Because of the cracked and patched condition of the pavement in 1958, it was impossible to evaluate the corner cracks. Indications were that corner cracks had been quite prevalent.

In 1950 there were 28 internal and 26 external corner cracks in all the 9-6-9 sections. In the 7-in. uniform sections there were 2 internal and 6 external corner cracks, whereas in the comparable 9-6-9 sections there were 3 internal and 5 external corner breaks. The difference between these comparable sections is scarcely significant.

Blow-Ups

The only blow-up on this project occurred in 1950, 10 yr after construction. This blow-up was located at an untied, keyed, construction header joint located 510 ft from the expansion joint at one end of a 1, 740-ft expansion interval section. A slight raising

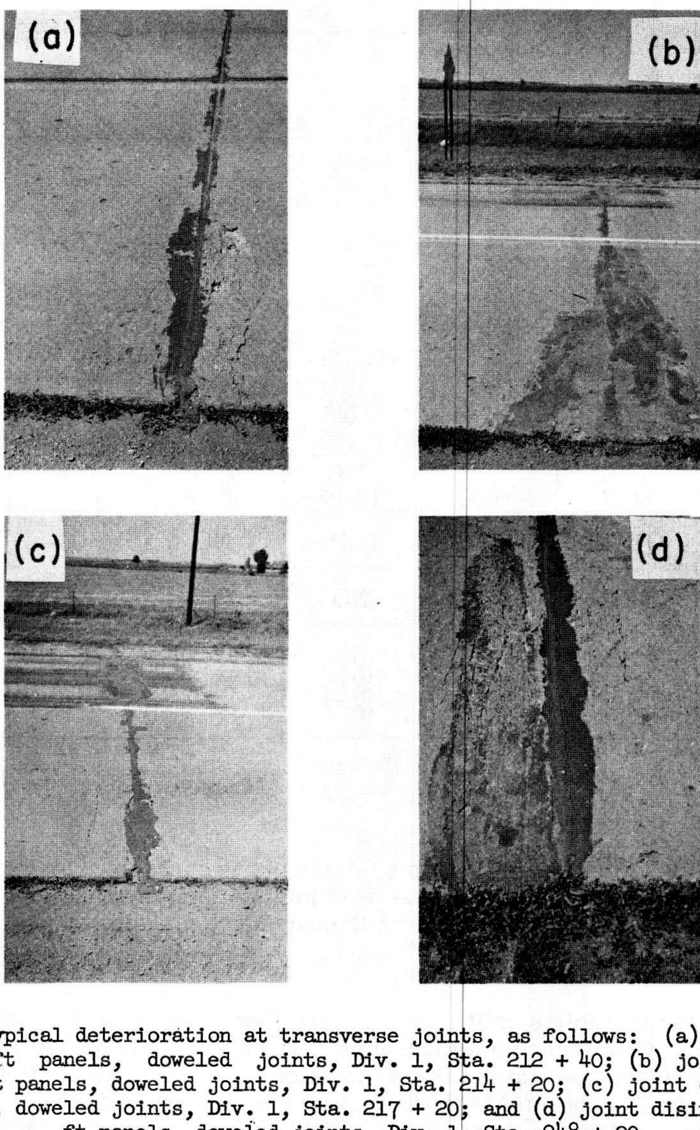


Figure 12. Typical deterioration at transverse joints, as follows: (a) joint disintegration, 30-ft panels, doweled joints, Div. 1, Sta. 212 + 40; (b) joint spalled and patched, 30-ft panels, doweled joints, Div. 1, Sta. 214 + 20; (c) joint disintegration, 30-ft panels, doweled joints, Div. 1, Sta. 217 + 20; and (d) joint disintegration, 20-ft panels, doweled joints, Div. 1, Sta. 248 + 90.

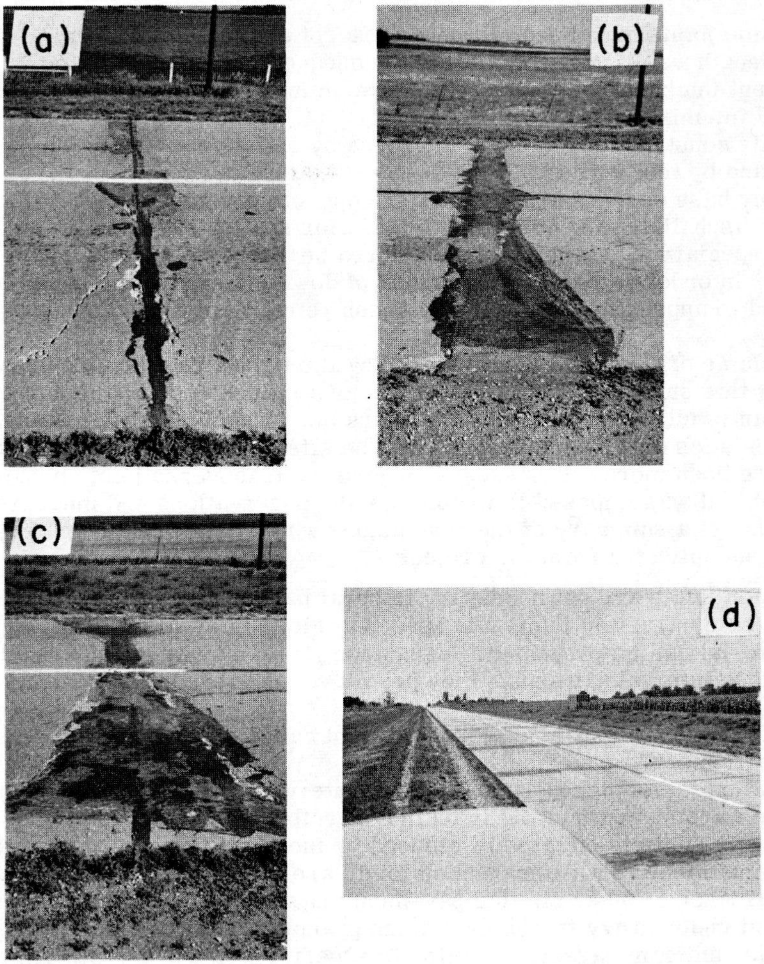


Figure 13. Typical deterioration at transverse joints, as follows: (a) joint deterioration, 15-ft panels, no dowels, Div. 8, Sta. 379 + 80; (b) expansion joint between Div. 8 and 9, Sta. 400 + 50, 25-ft panels to the left, 15-ft panels to the right; (c) patched joint, 25-ft panels, no dowels, Div. 9, Sta. 414 + 45; and (d) 30-ft reinforced panels, all joints doweled, Div. 17, Sta. 612 looking west.

of this joint had been noted in 1944, indicating the possibility of the failure which eventually occurred.

A contraction joint, located 630 ft from the aforementioned blow-up in a 1,245-ft expansion interval section, had also shown partial failure due to compression. About 3 ft of the concrete on one side of this joint had disrupted in the manner typical of blow-ups.

Since 1950 the disintegrated condition of the concrete in the vicinity of many of the joints would tend to relieve compressive stresses and eliminate the common type of blow-up.

CONCLUSIONS

The study of the Minnesota experimental concrete pavement project has contributed much toward the understanding of certain problems related to concrete pavement design and performance. Observations made on the project have influenced a number of changes in pavement and subgrade design in this state such as the following:

1. Expansion joints have been eliminated except at bridges.
2. Panel length was reduced to 15 ft but subsequently increased to 20 ft.
3. Pavement thickness was restored to 7 in. minimum and more recently has been increased to 8 in. minimum.
4. Subgrade construction has been improved by soil selection, moisture and density control, and by test rolling.
5. Granular base courses are used under concrete pavements.
6. Rubber-asphalt sealing compound is used for sealing pavement joints.
7. Where special load transfer is considered necessary at joints, dowel bars are provided; and, in order to reduce the number of dowelled joints, the panel length has been increased to approximately 40 ft with mesh reinforcement.

The low volume of commercial traffic during the first 8 yr after construction was a limitation of this study which may have had more influence on the performance of the project than is fully appreciated. Had there been heavier traffic loadings, the structural characteristics of the various designs, the effects of the subgrade, and other factors might have been more fully assessed before the transverse joints became spalled to the extent that it was impossible to continue the observations and measurements.

The following is a summary of the conclusions which have been indicated by the data and observations collected from the project:

1. Expansion joints are not necessary in rural pavements, except at fixed objects, provided that the contraction joints are spaced at close intervals and that infiltration of foreign material can be prevented. Expansion joints may be considered detrimental if placed at close intervals, because they permit excessive joint movements by panel migration.
2. The elimination of expansion joints will not cause excessive compressive forces in the main portion of the pavement.
3. Contraction joints placed at 15- to 20-ft intervals will control transverse cracking and reduce warping and faulting in plain concrete pavements.
4. Faulting at joints is effectively reduced by means of dowels. Aggregate interlock may also be effective when expansion joints are eliminated and short panels are used. (Performance of other similar pavements has shown that aggregate interlock is not sufficient under heavy traffic even when granular bases are provided.)
5. The 7-in. uniform pavement section is superior to the 9-6-9 section in resisting longitudinal cracking in plain concrete pavement, although neither section is adequate for heavy commercial traffic.
6. Reinforcing steel (longitudinal bars or mesh) will not aliminate transverse cracking, but will prevent the cracks from opening. Wire mesh reinforcement is effective in controlling longitudinal cracking in 60-ft panels.
7. Spalling in the joint areas is associated with restrained concrete, metal seals and asphalt-diatomaceous earth joint filler. However, it is felt that pavement deflection resulting from inadequate subgrade support is a major contributing factor to spalling.
8. Rubber-type joint fillers are superior to asphalt-diatomaceous earth joint filler, but their effective life is limited when not properly maintained. Extended postponement of joint maintenance is detrimental to the pavement.
9. Expansion joint fillers, where used, should be non-extrusive in service and should prevent leakage of water downward to the subgrade.
10. Subgrades of high and uniform density accompanied with a gravel base are desirable to reduce longitudinal cracking and improve the general performance of the pavement.
11. Concrete pavements tend to become rougher with age due to the combined effects of climate, subgrade, traffic and gradual deterioration of the concrete.

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European Developments in Prestressed Concrete Pavements for Roads and Airports

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This is a general review of developments in the field of prestressed concrete pavement in England, Germany, Belgium, Switzerland, Spain and Italy, as compared with French developments in the same field. The paper concludes with a brief economic study based on data collected during the tests on experimental slabs at Orly, together with fairly complete descriptions of the tests themselves.

● IN A PERIOD where prestressed concrete is undergoing considerable developments in the United States and tests on prestressed pavements are being carried out in various American laboratories and on test strips, it may be of value to make a survey of European experiments in this field.

Preliminary information on the subject was presented by the author at the International Concrete Road Conference at Rome in 1957. In this report more recent work and particularly the projects carried out in 1959 in Germany, Belgium and France are described.

ENGLAND

The lecture delivered by James Pearson Stott, in charge of research work at the Road Research Laboratory of the Institution of Civil Engineers on March 1, 1955, gives a detailed account of the tests carried out in Great Britain up to 1954.

The author obtained further details during a visit to that country some time ago. The tests carried out in Great Britain are given in Table 1.

Tests were performed on individual slabs, prestressed by longitudinal cables without transverse prestressing or by diagonal cables symmetrically arranged in two directions in relation to the center line of the slabs, or by longitudinal and transverse cables at tight angles to each other.

The South Benfleet test was particularly interesting on account of the fact that after the road was constructed, it became necessary to cut it open thus eliminating the longitudinal prestress for a time. This experience showed that it is not much more difficult to repair a prestressed concrete road than an ordinary concrete one.

GERMANY

The earliest prestressed concrete paving project in Germany was the tank testing platform constructed for the French corps of engineers at Spire in 1954. This platform consists of a series of slabs 50 to 70 meters long (164 to 229 ft). The pavement has a total thickness of 20 cm including a 14-cm subcourse and a 6-cm wearing course. The slabs are laid on a 30-cm river gravel base course with a 2-cm fine sand topping. A layer of Kraft paper was placed between the sand and the concrete pavement. The total length of the platform is 1,500 meters (4,900 ft).

The prestress system used is of the Leoba type with a stress of 30 tons per element. Each unit consists of 6 steel cables the extremities of which are provided with threaded rods and bolts by which they may be placed under tension.

TABLE 1

Date	Site	Length (ft)	Width (ft)	Thickness (in.)	Prestressing Longit.	Prestressing Tran.	System of Prestressing	Present Condition
1950	Crawley, Sussex	393	19	6	210 psi	22.4	Diagonal cables	Good
1951	Wexham	108	10	6	182	-	Long. cables only	Good
	Springs	127	10	6	126	-	Long. cables only	
	Buckinghamshire	196	11	6	252	-	Long. cables only	
1951	London Airport	328	118	6.3	560	560	Triangular slabs, Orly type	Good
1951	John Laing's Ltd	50x164= 8,200	11	6	294	-	Longitudinal cables	Good
1952	Basilton, Essex	13x164= 2,232	15	6	280	-	Longitudinal cables	Good
1952	Woolwich	3,280	16-26	6	252	28	Diagonal cables	Good
1954	Port Talbot	1,476	19	6	280	Varying	Longitudinal diagonal cables with jacks	No information
1954	South Benfleet	328	19	4	532	49	Flat jacks in longitudinal cables in transverse direction	Good after reinstatement of the longitudinal compressio

The total sectional area of the 6 cables consisting of 12 strands is 2.65 cm^2. Stress is equal to 80 percent of the elastic limit. The slabs are separated by transverse joints with bituminous coated dowels at the base. The upper part of the joints is filled with a special joint compound.

This platform has been in use since 1954 and no appreciable wear has been noticed to the present time. Because of the length of the slab elements, the joints are several centimeters wide, however, and variations in the width of the joints due to differences in temperature cannot be taken up by the joint compound.

This condition is not serious as far as tank traffic is concerned but a certain spalling of the joint edges has resulted. Transverse prestress units exist only near the joints. On the whole the using services are satisfied with the manner in which the platform has stood up under exceptionally heavy traffic.

Various tests performed in Germany in 1954 were described in the reports of the Concrete Pavement Committee of the German Pavement Association. Three systems were tested. The first, used in the slabs constructed at Mergelstetten I and II, included both longitudinal and transverse prestress cables. The slabs were 120 meters long (393 ft), 8 meters (26 ft) wide and 15 cm thick. Longitudinal prestress values were 22 and 17 kg/cm^2 (308 and 238 psi transverse prestresses 8 and 3 kg/cm^2 (112 and 42 psi). The fact that pavement continuity was interrupted at 120-meter (393-ft) intervals made it necessary to provide special and costly cable holding devices and did not avoid the joint problem. At Mergelstetten III cable prestressing was also used but the cables were placed diagonally, thus partly avoiding the joint problem. Cable anchorages were set in prefabricated lateral strips which were necessarily costly.

As a result of these tests, the government decided to construct a by-pass at Bingen with a 1,000-meter (3,280-ft) prestressed concrete section). Design specifications provided for a very wide safety factor so that the pavement would stand up under any circumstances. The section is overdesigned. The prestressed pavement is 16 cm thick; longitudinal prestressing is 50 kg/cm^2 (700 psi) with a total tension of 65 tons per cable; transverse prestressing is 15 kg/cm^2 (210 psi) with a total tension of 9 tons per transverse bar.

The slab is placed on a 3-course hot-laid bituminous concrete foundation 15 cm in thickness. The upper course of the foundation has a very smooth surface and is composed of sand compacted in a bituminous binder. To reduce friction, the bituminous concrete was covered with two layers of talc-coated paper before the prestressed concrete was placed. The coefficient of friction was thus reduced to 0.35 instead of 0.8, the figure on which design was based. In places, the paper is replaced by plastic sheets which give the same results.

An "antifrost" base was provided under the bituminous concrete foundation. This base is 40 cm thick and consists of sand and gravel compacted by a vibrating roller to 98 percent of the Modified AASHO optimum corresponding to a dry density of 2.4 (that is, an extraordinarily high degree of compaction).

This base was placed on compacted unclassified Rhine Valley fill material.

The section consists of 150-m (492-ft) long slabs separated by steel and rubber joints similar to those described in HRB Proceedings, Vol. 37, pp. 150-193.

Longitudinal prestress cables consist of jack-prestressed strands in thin sheet metal tubes. Transverse prestressing is provided by 7.50-meter (30-ft) bars fixed at one end and prestressed by jacks at the other. Prestress tension is maintained by a bolt which is tightened.

The by-pass at Bingen is in service at the present time. The plate-and-rubber joints appear to be standing up well and to be very slightly affected by traffic.

At the same time, under the influence of organizations grouping highway research specialists and concrete and cement technicians, prestressed concrete made great strides in the field or airfield runway construction.

In 1956 a test runway was constructed at Memmingen by the contracting firm of Dyckerhoff and Widman. It consists of 100-meter (328-ft) slabs, 7.70 meters (30 ft) wide and 14 cm thick with an increased thickness at the edges bringing the total thickness to 20 cm. The ends of the slabs are spaced at a distance of 1 meter and the slabs are prestressed by single cables 10.2 mm in thickness. Prestressing is both longitudinal and transverse. The transverse cables cross the entire 45-meter (147-ft) width of the runway. A special system was provided in the spaces between the slabs in order to insure satisfactory tensioning of the cables. After the cables have been tensioned, concrete is placed so that a space of only 3 cm is left for longitudinal movement of the slabs. The cables are inside metal tubes whose diameter is only slightly greater than that of the cables.

The tubes are left in the concrete at the time it is placed. The cables are then introduced into the tubes and tensioned. The space between the cables and the inside of the tube is then filled with grout injected so as to provide a strong bond between the cables and the slab.

The success of this runway led the contractor to include a prestressed concrete alternate with his bids for construction of runways at Wunstorff, Hopsten and Nordholtz. The alternates were accepted in all three cases as being both more economical and technically more satisfactory.

The proposed project was to be a bituminous concrete runway on a 20-cm stabilized macadam foundation. The ends of the runway were to include 300-meter (984-ft) long sections of concrete 22 cm thick on a stabilized macadam base. The alternate proposed by Dickerhoff and Widman was a prestressed concrete runway 14 cm thick and 20 cm thick at the outer edges. The pavement was to be placed directly on a 45-cm thick anti-frost sand base with a 2-cm thick fine sand topping covered with a double thickness of paraffinned paper. Elimination of the stabilized macadam foundation (materials for which had to be transported from a great distance and which the contractor found superfluous if not actually harmful when used under a prestressed concrete slab) made up for the difference in cost between prestressed and ordinary concrete. The total cost per square meter of prestressed concrete was 27 DM as compared to 22 DM for a 22-cm ordinary concrete slab. The saving on foundation costs largely compensates the 5 DM difference.

Because the slab rests directly on sand, compaction of the latter was carried out with particular care. Vibrating caterpillar compacters were used and, at Hopsten, with a fine, slightly silty sand, a modulus of reaction of 14 kg/cm^3 was obtained (that is, an extremely high value).

The critical point was to find a method of construction which would insure pavement continuity between successive slabs and permit longitudinal cable prestressing and movement of the slabs caused by variations in temperature after prestressing. Calculations indicated that a space of 6 cm would be required between slabs. Experience made it possible to reduce this width to 3 cm.

In addition, for longitudinal cable prestressing, jacks had to be placed between successive slabs and tension applied so that the required displacement could be obtained. The ends of all cables were stamped to provide a projecting screw thread, the diameter

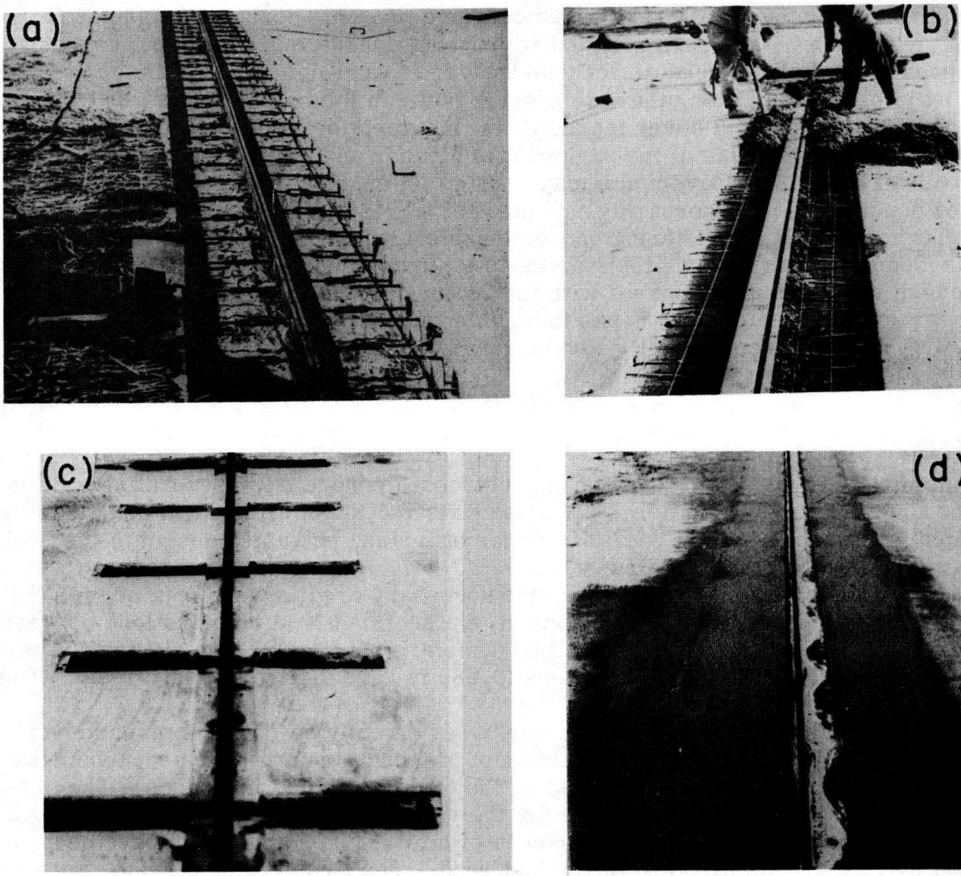

Figure 1. Hopsten Airport, Germany. Stages in construction of a joint using the Dywidag method: (a) cables placed and connected to central steel railing; (b) concrete hand placed in joint leaving room for jacks; (c) joint ready for jacking; (d) finished joint.

of the groove being equal to that of the cable. This made it easy to fix the cables to the jacks and is one of the patented features of the Dywidag system (Fig. 1).

Placing the jacks, supporting them against fresh concrete surfaces with the use of metal elements to provide the required 3-cm joint space, placing metal joint covers, fixing cables and filling the space between slabs with concrete would, at first, seem to be a rather complicated series of operations but it is claimed that the work proceeds automatically when a trained crew is employed. In any case, it is certain that the company using this method of construction has already built a two-section, 1,276-meter (3,828-ft) long runway at Wunstorff and a 3,000-meter (9,842-ft), 30-meter (98-ft) wide runway with a parallel taxiway of the same length at Hopsten (that is, a total prestressed concrete paving area of 170,000 m^2. The firm has now begun design of installations at the Nordholtz airfield near Cuxhafen. Construction will include a 2,900-m (9,514-ft) long, 45-m (147-ft) wide runway and additional paved areas bringing the total surface to 260,000 m^2. These are NATO fields and pavements and are designed for 20-ton wheel loads. The thickness thus remains at 14 cm. Concrete will have a compressive strength of 450 kg/cm^2, (6,300 psi) and a tensile strength of 60 kg/cm^2 (840 psi). Rhine valley sand and gravel will be used for aggregate as well as crushed rock. An air-entraining agent and a plasticizer will be used as is customary. Under these circumstances, four 130- by 7.50-meter (426-ft by 29-in.) slabs can be placed per 12-hr shift which represents 450 m^3 of concrete placed per day.

In addition to the fact that prestressed concrete presents certain advantages from the point of view of cost, the users feel that it presents considerable technical advantages. A thin prestressed slab placed directly on a layer of sand is highly elastic and is preferred by pilots. In addition, the wide spacing between joints and the fact that they are provided with a metal joint cover makes them imperceptible to the pilot as he runs over the pavement. The surface of the pavement is cleaner and free of loose gravel which, in the case of bituminous pavements may damage jet engines on group flights. Prestressed concrete is now thoroughly accepted for use in airfield pavement construction in Germany as a technique offering unquestionable advantages. The Dyckerhoff and Widman Company plan to apply their methods to highway construction. The reduction of pavement thickness as compared with the 14 cm thickness of runway pavement cannot be very great, but the company hopes to be able to reduce thickness to 10 cm and, at this figure, to provide an economically advantageous alternate.

BELGIUM

Test Road

Belgium has shown a keen interest in prestressed concrete for several years. In 1957 the Centre d'Etudes Routières proposed construction of a test road and, with the agreement of Hondermarcq, General Director of Roads, entrusted design of the project to Paduart of the University of Brussels.

The project was designed in 1958 and the necessary credits were obtained for the fiscal year 1959. The test road will consist of a section 3½ kilometers long between Zwartberg and Meeuwen. The road will be 7 meters (26 ft) wide and will include two curves with radii of 1,000 and 500 meters (3,280 ft and 1,640 ft), respectively. The terrain is practically horizontal.

The characteristics of this test road are based directly on the experimental work conducted at Orly in 1957, that is, a minimum of foundation and pavement thicknesses fixed at 8, 10 and 12 cm as was the case in the tests conducted in France. The prestress system is the same as that used for the runway at Maison-Blanche (that is, flat jacks in active joints spaced at 270-m intervals). Fixed abutments have been designed by Paduart for each end of the test road. The first will be a reinforced ordinary concrete slab anchored by 80-cm spuds driven into the ground. The other abutment will also be a thick reinforced concrete slab anchored in the soil by six thin parallel fins, each 1 meter high and 15 cm wide (Fig. 2).

The most original feature of the project is the system for preventing relative movement of the slabs at the active joints. These joints are constructed over tunnels 1.70 meters deep which may be used for cable and utility crossings.

The slabs rest on the vertical supports of the tunnels. Upward movement of the slabs is prevented by bars grouted into the concrete at the bottom of the tunnel and bent and imbedded in the concrete of the road pavement slabs. These bars are free-standing for their entire height and permit slight horizontal movement of the slabs through their elasticity.

This extremely original and very simple solution would appear to give results comparable to those obtained by the more complicated system adopted at Maison-Blanche.

The soil on the test road site is a slightly silty sand, easily compacted and of good quality. This sand will be used for the foundation. It will be compacted for a width of 8 meters, waterproofed by spreading cutback and covered with 2 cm

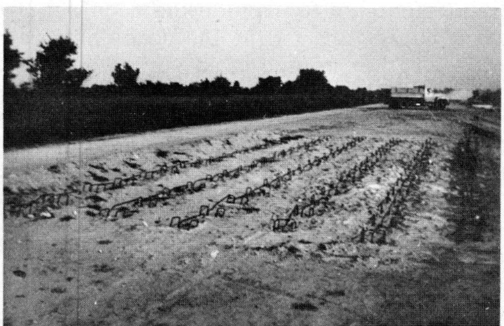

Figure 2. Test Road between Zwartberg and Meeuwen, Belgium: One of the abutments—fins poured and reinforcements placed. Slab will have to be poured afterwards.

of sand to facilitate movement of the slabs. The thin layer of sand will be covered with Kraftpaper on which the concrete will be placed.

The designers have thus reduced foundation thickness to a minimum. Inasmush as the soil is not susceptible to frost and can carry truck and heavy equipment traffic in all weather conditions, this solution would appear to be a satisfactory one.

The pavement consists of 8-, 10- and 12-cm thick slabs prestressed by jacks placed in the active joints (as previously described).

Different prestresses will be adopted for longitudinal jack prestressing and for transverse prestressing. The latter may be replaced by reinforcement or eliminated altogether. A total of 26 sections with different characteristics are planned. Up to the present time, only the abutments and the tunnels for placing active joints have been constructed. The remainder of the work was completed in the winter of 1960.

Runway at Melsbroek

Some time after design was begun on the test road, the management of the Melsbroek Airport near Brussels decided to invite bids for construction of a 3.3-km runway 45 meters in width designed for jet planes.

The runway was to be constructed of ordinary concrete. Bidders learned, however, that they could present alternate designs for prestressed concrete. Paduart, who had just finished design of the test road, accordingly proposed construction of a prestressed concrete runway to be executed in collaboration with the Wegebo Company.

The ordinary concrete runway was to consist of a slab 45 cm thick composed of two courses 22.5 cm in thickness, the lower course to be 325-kg concrete and the upper 400-kg concrete.

The base was to consist of a layer of 0.60 quarry run aggregate laid on the natural soil, a sandy silt. The alternate proposed by Paduart was a prestressed 18-cm slab placed on a 2-cm layer of sand above a 20-cm base course. The upper 10 cm of the base course was to be 0.60 quarry run aggregate and the lower 10 cm compacted sand.

The prestressed concrete proposal offered a saving of approximately 5 percent as compared with ordinary concrete and was selected by the Airport authorities. Work began in the spring and the completed runway was opened to traffic on 15 December; that is, after 6 months work. Results seem, at the present time, to be quite remarkable. Five of the six 7.50 strips have already been placed for the entire 3.3-km length. Practically no cracks are noticeable and the joints covered with concrete will be imperceptible when completed.

Certain features designed by Paduart which were to be tested on the test road before being used in permanent construction were used on the Melsbroek runway before being tested. It would appear, however, that the conditions under which they were designed make it possible to anticipate that they will stand up well in use.

The original features which were described in connection with the test road have been used for the Melsbroek runway. The abutments are of the ribbed type and are ordinary concrete slabs 40 cm in thickness connected by reinforcement to fins 40 cm in width and 1.20 m in height measured from the lower surface of the slab. The ribs were placed without forms in the natural soil which had been excavated by a specially designed machine like a mechanical trencher. The soil was sufficiently cohesive to permit trenching without shoring and placing vibrated concrete without caving in of the trench walls.

Contraction joints 5 cm deep have been provided in the upper part of the abutments.

The abutments are extended by large ordinary concrete slabs which connect the main runway to the taxiways. In the event that the abutments should prove insufficient to absorb expansion of the runway in warm weather, measures have been taken so that they can be reinforced by uniting them to these end slabs.

The active joints above utility tunnels described in connection with the test road are also found at Melsbroek (Fig. 3). Slab thicknesses have been increased 18 to 25 cm at these joints. Active joints are spaced at 330-m intervals (1,082 ft). Temporary joints were placed every 110 m (360 ft) following the example at Maison-Blanche. These joints make it possible to compress the concrete 24 hr after placing and it is because of them that the 110-m long slabs show no fissures due to contraction.

Transverse prestressing will be provided by cables spaced at 1.75-m intervals. Each cable will consist of twelve 7-mm strands tensioned at 100 kg per mm^2 for a breaking strength of 150 kg. Cables will be placed directly in the slab without tubes. Passages for the cables were provided by setting pipes on concrete blocks at the time the slabs were poured.

These pipes were removed when the concrete had sufficiently set.

The different strips which go to make up the runway were successively placed, each of them serving as a form for the following one. It is anticipated that the fissures between the strips which were caused by contraction will be eliminated at the time of transverse prestressing.

At Masion-Blanche, the longitudinal joints were protected by strips of paper before transverse prestressing. The contractor at Brussels expects to obtain similar results by scraping out the interior of the fissure with a knife before applying compression.

The foregoing are the chief features of the Melsbroek runway. When opened to traffic, it will be the largest prestressed concrete runway in operation. It will be extremely interesting to see how it stands up in a climate which is appreciably different from that of Algiers.

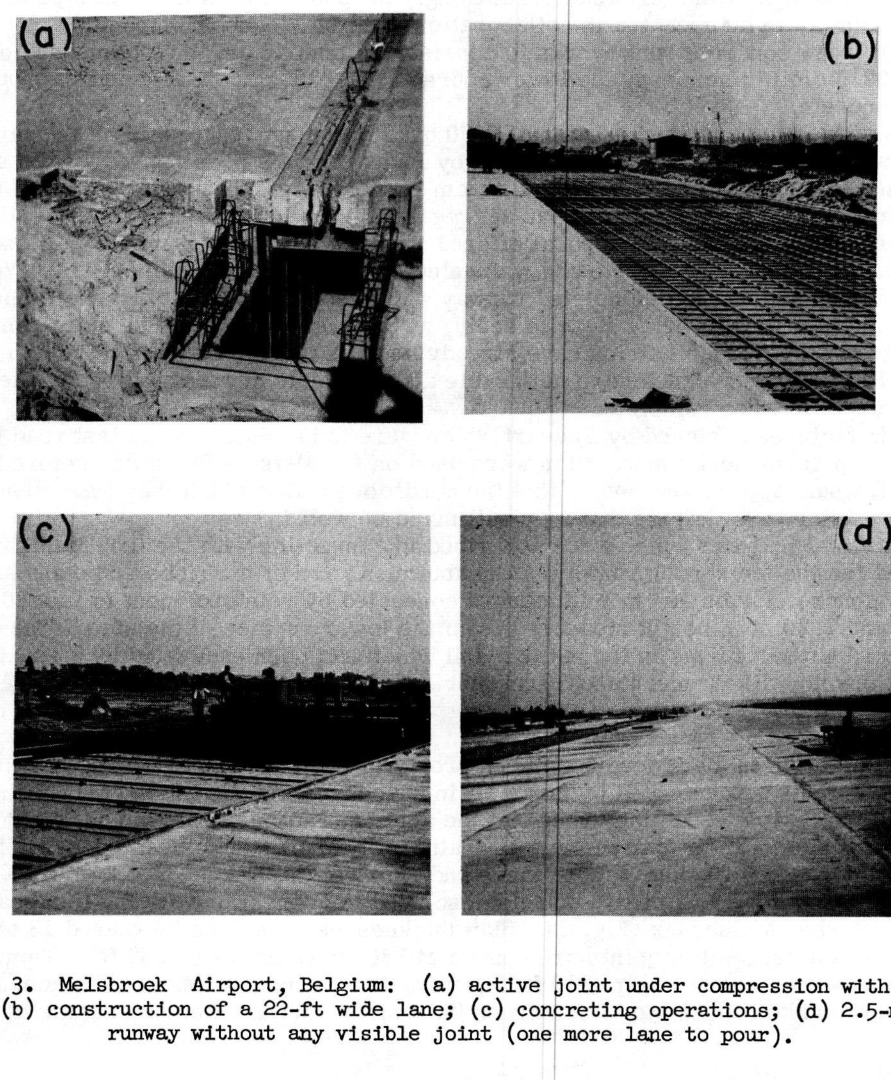

Figure 3. Melsbroek Airport, Belgium: (a) active joint under compression with underpass; (b) construction of a 22-ft wide lane; (c) concreting operations; (d) 2.5-mi long runway without any visible joint (one more lane to pour).

Experimental Taxiway at Melsbroek

A prestressed concrete test taxiway section was constructed at Melsbroek in 1958. This section is 350 m (1,148 ft) long and 23 m (75 ft) wide and 10 cm thick. The pavement rests on a 30-cm layer of compacted sand above the natural soil compacted for a thickness of 30 cm. The pavement consists of prefabricated slabs in the form of parellelograms 12 m (39 ft) long and 1.24 wide. These slabs are prestressed by grouted strands at the time they are manufactured. They are joined together in 18 strips to make up the total width of the pavement. These slabs are set directly on the sand base course and held together by transverse cables consisting of eight 7-mm strands. The cables are spaced in openings provided in the slabs at 80-cm intervals. Each cable exerts a pull of approximately 26 metric tons. Transverse prestressing is approximately 33 kg. The oblique joints in the slabs are staggered from one strip to the other so that each of the joints is crossed by one cable. Because the joints are oblique, the transverse stress transmitted by the cables gives a component perpendicular to the joints thus insuring a longitudinal prestress bond between the butt ends of the slabs. There has not been sufficient time to determine how this taxiway section will stand up under traffic.

The fact that the Administration did not accept this system for the construction of the Melsbroek runway would appear to be an unfavorable indication, however.

SWITZERLAND

The first two tests made in Switzerland by the Concrete Road Association included two sections each 500 m (1,640 ft) long. In the first of these, longitudinal compression is obtained by means of flat jacks placed every 70 m (229 ft) with a fixed abutment at one end and a natural rock abutment at the other. In the second section compression was obtained by means of wedges placed at 120-m (393-ft) intervals. One end of the slab is held by a fixed abutment, the other end abuts against an existing road. The longitudinal prestresses are quite high : 60 kg/cm^2 and 70 kg/cm^2 (840 and 980 psi), respectively.

On the other hand, the transverse prestress has a low value. In the first case it is produced by single strand cables (7 mm diameter) 1 m apart and tensioned so as to produce an initial compression of 4 kg/cm^2 in the slab. In the second case there is no transverse prestress but the slab is provided with light reinforcement. It should be noted that in both cases the slabs are narrow; namely, 2.5 m and 5.5 m wide, respectively. British experience has shown that in such circumstances transverse prestress may be eliminated (these two slabs form part of a concrete country road). An interesting feature of the second test is that the slab included a curve in the road; this curved portion has behaved well up to now.

SPAIN

An experimental road is to be built in the Madrid area in the course of 1960 under the supervision of Escario, Professor of Road Technique at the Technical College for Bridges and Highway in Madrid. The necessary credits have been put at his disposal. This road will include a prestressed concrete section more than 1,000 m (3,280 ft) in length. The methods which will be applied are still under study.

ITALY

A part of a road has already been constructed under the supervision of Lévi and was the subject of a special report at the International Concrete Road Congress at Rome in 1957.

FRANCE

Early Experiments

French tests on prestressed concrete roads were conducted by Dollet, Chief Engineer of Bridges and Highways. He was responsible for the construction of slabs at

Luzancy and at Esbly while he was Engineer at Meaux. Later he built the experimental road at Bourg-Servas, in the department of Ain, where he is now Chief Engineer.

In its issue of January 1954, the review "Travaux" devoted an article to the experimental road at Bourg-Servas. The authors of the article were Dollet and Robin. The main points are summarized here.

At Luzancy there are two slabs, 6 m wide and 24 and 26 m long, respectively, forming two sections of National Highway 369 between La Ferté sous Jouarre and Château-Tierry. These slabs are 16 cm thick at the center line and 20 cm thick at the edges.

The prestress is obtained by means of cables placed at an angle of 45 deg to the center line of the road. These cables consist of 10 strands of 5-mm diameter and are arranged to form 50-cm meshes. The prestress varies from 17 to 21 kg/m^2.

These slabs were constructed in 1945-46 on fill at both approaches to the Luzancy bridge. Tests carried out on a 16-cm thick prestressed concrete slab at Orly at the same time showed that it could support 130-ton load distributed over a circular area 70 cm in diameter. Of course, roads are never designed for such heavy loads. The Luzancy slabs were therefore well on the safe side. They have stood up perfectly in spite of the low bearing capacity of the subgrade.

The slab at Esbly, which was constructed in 1949 on the heterogeneous backfill on the left-bank abutment of the Esbly bridge, is 48 m (157 ft) long and 15 cm thick. It was constructed between two precast curbs which hold the anchorage cones. The prestress is insured by a grid of cables composed of 12 strands 5 mm in diameter forming 1-meter meshes. The prestress is 16 kg/cm^2 (224 psi). In spite of poor subgrade conditions, this slab has also behaved remarkably well since its construction.

Encouraged by these two successes Dollet, Chief Engineer at Bourg, decided to carry out a test on a larger scale on a section of National Highway 83 between Bourg and Lyon. The length of this experimental section was originally to be 1,000 m (3,280 ft) but its length had to be reduced to 300 m for budgetary reasons.

To preserve the uniform appearance of the road it was decided at the outset that the prestressed concrete should be covered with a bituminous surfacing 4 cm thick. The presence of this surfacing has prevented long-term observations and somewhat detracted from the value of this otherwise extermely instructive test.

The essential difference between the Bourg-Servas road and the slabs at Luzancy and Esbly is that the longitudinal prestress was produced by means of flat jacks instead of cables. This meant that abutments had to be provided to prevent movement at the ends of the slab.

It is not necessary to go into detail about the features of this project and about the abutment system which was adopted. The experience gained in this connection at Algiers has yielded data which would suggest that minor modifications could be made.

In any case the Bourg-Servas road was the first in which the longitudinal prestress was obtained without cables and consequently much more economically. This test held out a prospect that it might some day be possible—perhaps under special but nevertheless frequently occurring conditions—to construct a prestressed concrete road whose cost would not differ from that of an ordinary concrete road, the thickness of the slab being designed to obtain comparable strength.

The design of the Bourg-Servas road was prepared by Freyssinet and the Société Technique pour l'Utilisation de la Precontrainte (STUP). The information furnished by this test proved of great value in the design of the airfield runway at Algiers.

Present Status: Slabs at Orly

The remarkable success of the prestressed concrete runway at Algiers led the author to take up the question of road tests in the light of the results obtained. It seemed that the only economical method of obtaining longitudinal prestressing was to use "active joints" containing one or more flat jacks, and to use cables or bars for transverse prestressing. The question of the abutments cannot be investigated experimentally as long as the final location of the road has not been decided. Local conditions should be used in the best possible way; advantage should be taken of areas where a strong subgrade is available or of a curve where the natural configuration of the terrain can take up the horizontal thrust, so as to obtain the most economical abutment in each particular case. Therefore no tests were made on this point, but a fixed "frame" was provided in which the slab unit to be constructed would be enclosed. This frame was also made of prestressed concrete, the two short sides acting as abutments.

The question of the spacing of the active joints had been solved in Algiers, where these joints were spaced at 300 m (984 ft).

Actually, temporary joints were placed at every 100 m. Twenty-four hours after the slab had been concreted, compression was applied at these joints to prevent the formation of shrinkage cracks. After the concrete had set, the temporary joints were packed solid and the continuity of the slab was restored before applying compression to the actual active joints. The experience gained in Algiers showed that the stresses in the slab which were at first concentrated at the active joints were gradually distributed and that the slab as a whole acted as if the friction of the concrete on the sand base were zero for extremely slow movements such as those due to temperature variation.

Provided that the slab is laid on an appropriate sand base, the active joints can easily be spaced at 200 or 300 m (656 or 984 ft) as in Algiers.

An important feature of the tests is the design of the base. One of the chief advantages of prestressed concrete is the absence of joints, which secures the continuity and the impermeability of the slab. If a prestressed slab of practically unlimited length is compared with an ordinary concrete slab without dowels, it will be noted that in the first case the slab acts as if the loads were always applied at the center or, at most, at the two edges of the road, but never at a corner. The function of a road foundation, that is, to transmit loads acting on the surface of the slab to the natural subgrade so that only elastic deformations are produced, is thus reduced by one-half or even by three-quarters. It was assumed (and the object of the test was to check this) that the layer of sand indispensible to permit traffic of the contractor's equipment on the job was sufficient to support the prestressed slab and to prevent permanent deformations of the subgrade.

The thickness adopted for this layer of sand was 15 cm on account of the moisture content and the poor quality of the soil, a clayey silt with a CBR value between 1 and 2. If the tests are confirming, a considerable saving can be effected on poor ground which will reduce the cost of prestressed concrete construction to nearly that of ordinary concrete or perhaps even below it.

Another point which is to be investigated is the question of slab thickness. Previous tests and the experience gained in connection with airfields has shown that a 16-cm thick slab could without appreciable permanent deformation withstand a practically unlimited number of 100-ton loading and unloading cycles applied to a circular plate 70 cm in diameter. On a road the loads acting on an area of similar size will never be more than 10 tons corresponding to a 20-ton wheel load, which is higher than anything yet existing in practice. It would appear that a slab 8 cm thick can withstand alternating loads of 10 tons. The question at issue was whether it was possible to level the surface of the sand base so as to permit the easy construction of a slab of 8 cm uniform thickness, and whether this thickness would be sufficient to accommodate the cables without weakening the slab.

Accordingly, it was decided to divide the test slab into three parts with thicknesses of 8, 10 and 12 cm, respectively. The base course was accordingly levelled with particular care and was finished to a tolerance of ± 1 cm. This presented no exceptional difficulties and merely necessitated careful compaction with a light roller. The prestressing was obtained by means of single strands of 10 mm diameter with a bituminous coating. Experience has shown that such wires, which are supplied on reels, have no tendency to curve once they have been straightened and that before tensioning they can be easily held in position during the placing of the concrete by supporting them on small concrete blocks or wire chair spacers. These wires behaved very well during the tensioning. Once the initial friction had been overcome, only a small force was needed to move the wires, which showed that the bituminous coating was uniformly distributed along the length of the wires and was satisfactorily performing its function as a lubricant. Besides, the special Freyssinet cones are easy to install and economical. Although an alternate using transversal bars with threaded ends had first been considered, it was found that wires were easier to transport than bars 11 m long and could be placed just as rapidly. The idea of the bars was accordingly abandoned.

Another object of the tests was to ascertain whether an anti-buckling device such as the interlocking system used at Maison-Blanche was necessary to prevent any lifting of the slab near active joints as a result of some assymetry in the arrangement of the joint. Of the two joints, one was of similar construction to those at Algiers, whereas the other consisted merely of an additional slab thickness and a sleeper beam under the joint.

Construction of the Slabs.—Construction which began in early April, was completed on June 15, 1957. It comprised the construction of the base, the frame, and finally the slabs themselves (Fig. 4).

The foundation was obtained by stripping the natural ground and spreading a base course consisting of a mixture of 0-60 mm sand and gravel obtained from the Seine.

Measurements for determining the modulus of subgrade reaction were carried out on the soil and on the base course after compaction with a light roller. The following values were obtained:

Subgrade	0.75 - 2kg/cm^3	(8 - 24 lb/cu in.)
Base course	1 - 1.2 kg/cm^3	(12 - 15 lb/cu in.)

It was obvious that the subgrade soil was very poor. Its quality certainly improved on drying after the rainy season. Measures have been taken so that it can be saturated again at the end of the test, however.

The enclosing frame was constructed in accordance with the design prepared by the STUP and was prestressed without any special difficulties.

Finally, concrete for the three slabs, with a total length of 50 m (164 ft), was placed inside the frame. The slabs were constructed so as to bear against the frame at their two ends only. Only the center slab, 20 m (65 ft) long and 8 cm thick was placed between two active joints. The other two slabs which are 15 m (49 ft) long and 10 and 12 cm thick, respectively, were placed between the abutments and the active joints.

Transverse prestressing was obtained by means of jacks, using special cones for single strand cables developed by the STUP. No difficulties were encountered except for the snapping of one cable which was replaced immediately.

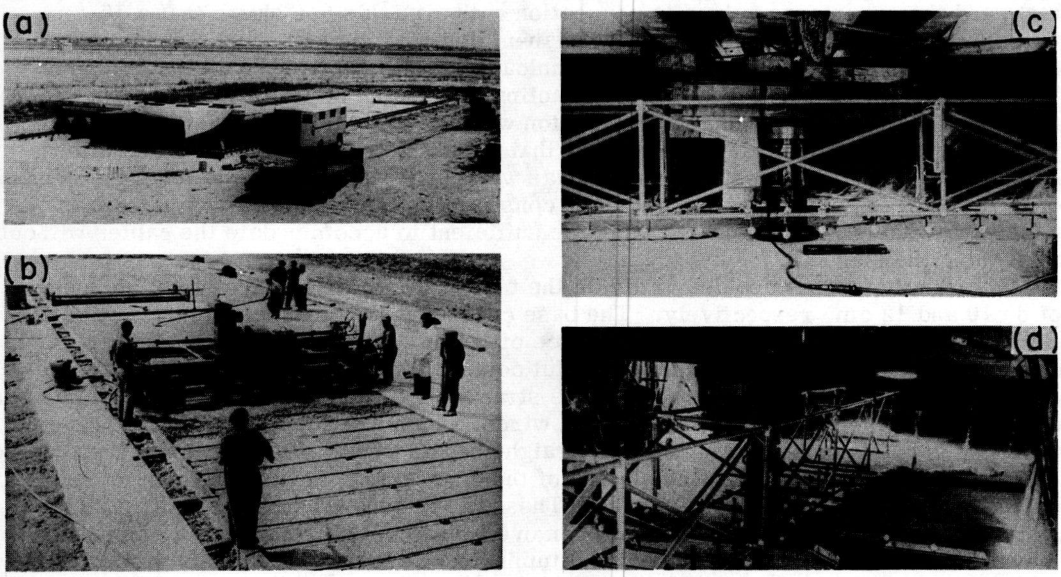

Figure 4. Slabs at Orly: (a) general set-up of test slab; (b) pouring slab—transverse strands placed; (c) testing set-up—repeated loading operations; (d) deflection measurement.

Once the initial resistance had been overcome, the bituminous coating proved to be effective and the frictional resistance became negligible. Longitudinal prestressing was effected by means of flat jacks which were inflated by hand after being first filled with water and subsequently with oil. In order to be able to continue the tests, the jacks will not be filled solid with grout as is usually done. After a few applications of compression, the joints were impervious and the pressure could be kept constant without difficulty.

Test Program.—Strength tests were started on July 4, 1957. The equipment used was described by Cot and Becker in articles on the 1953 tests which were published in "Travaux." The machine makes it possible to apply alternate loading and unloading cycles at a rate of 1,000 in a 6-hr period. The test is extremely severe because the load is constantly applied at the same point instead of being distributed in an irregular manner over the surface of the pavement as a whole, as it normally would be in the case of a road. It entails a risk of causing permanent deformation of the road base, whereas, in reality, there would merely be some settlement of the latter.

To begin with, 15-ton loads were applied with a uniform transverse and longitudinal prestress of 10 kg/cm^2 (140 psi) and the permanent deformations of the slab were observed. In reality, a load of 15 tons is more than 50 percent higher than the heaviest conceivable wheel load and is therefore amply sufficient.

In a second series of tests, the average transverse prestress was reduced to one-half its value by putting every alternate cable out of action, while maintaining a load of 15 tons.

In a third series of tests, the longitudinal prestress was increased from 10 to 100 kg/cm^2 (140 to 1,400 psi) which corresponds to a rise of 30 C in the temperature of the slab without a change in the moisture content. The transverse prestress was 5 kg.

Elasticity of the Slab and Deformations of the Subgrade.—At the very first test, a considerable difference was noted between the deformations of the pavement under load and the deformation remaining after unloading, thus demonstrating the well-known elasticity of prestressed concrete. While the deformation of a 10-cm thick slab was 2.3 mm under a load of 15 tons, after 100 tests, the residual deformation was less than 1 mm. Thus, the slab underwent an elastic deformation of about 1.5 mm. The foundation under the load should, like the slab, be increasingly deformed with the number of cycles to about 2 mm. But the subgrade has very little elasticity and should not recover its initial volume on unloading. A certain vacuum would tend to form under the slab if the latter continued to receive loads constantly applied at the same point. But in reality a road slab would receive loads distributed over the whole width of the pavement so that instead of a localized deformation the subgrade would undergo a uniform settlement, the slab constantly resting on the ground which would simply sink 2 or 3 mm.

The risk of vacuums forming between the slab and the ground is therefore, neither in the case of a road nor in that of an airfield runway, a real one, because the loads are distributed on the central strip which would settle uniformly under the effect of the compressions distributed over the whole surface. A wide safety margin should be allowed, however; that is, the deformations which lead in due course to uniform settlement should be strictly limited. It will be seen that the necessity of limiting loads at the edges of the pavement makes this safety margin imperative. But the risk of seeing vacuums form, as after the tests, between the slab and the ground is non-existent. The distribution of the loads will insure the uniform settlement of the base; that is, will maintain constant contact between the under surface of the slab and the ground.

Alternate Load Tests.—Tests were carried out on each of the 10-, 8- and 12-cm slabs, first at the center of the slab, then at a distance of 30 cm from the edge.

Longitudinal and transverse prestresses were both 10 kg/cm^2 (140 psi). The following loads were applied on plates 30 cm in diameter:

At center of slab	15 and 30 tons
At edge of slab	10 and 15 tons

The tests showed that 30 tons probably corresponds to the maximum load for the 8-cm slab. Deflection was more than 7 mm at the first loading and increased to 11.4

mm after 600 loadings. Residual deformations were 1.30 mm after the first loading, increasing to 2.5 mm after 600 loadings. It would appear that the loading limit was reached in this case. Because the maximum load admissible for a given slab is limited to what the slab can support at the edge, deformations at the center should remain very slight.

Figure 5 shows the comparative deformations of the 8-, 10- and 12-cm slabs for a load of 15 tons applied at the center; Figure 6, for a load of 10 tons applied at the edge. Both loads were chosen as corresponding to linear deformations as a function of the load. It is seen that in both cases the extent of the deformations is the same; that is, at 30 cm from the edge the bearing capacity of the slab is approximately two-thirds of what it is at the center. The diagrams also show that the tests corroborate the observations previously made by the Services de l'Aéroport de Paris concerning the nature of deformations of prestressed slabs. When these deformations are expressed as a function of the number of loading cycles on a logarithmic diagram, it is observed that the deformation varies as the logarithm of the number of the cycles. This observation obviously only holds true for relatively weak loads, the repetition of which produces progressive settlement of the subgrade. In the case of heavier loads, stabilization of the subgrade is reached more rapidly. This is of interest inasmuch as the observation of less than 100 loading and unloading cycles makes it possible to plot a complete curve of the deformations.

As can be seen, even in the case of the 8-cm slab, a concentrated load of 10 tons applied on a surface so that the pressure exerted is 10 kg/cm^2 (140 psi) does not produce residual deformations greater than 1 mm for about 1,000 cycles. When the load is always applied at the same point, the deformations under load increase from 2 to 3.7 mm.

Mention has already been made of the extent to which this test situation is an artificial one. As, in the point of fact, loads would actually be uniformly distributed on the slab, there would be no deformation but a uniform settlement. For an 8-cm slab with a load of 10 tons, which seems permissible, this settlement would reach approximately 2.5 mm at the end of a very long time.

Calculations have been made based on the results of the foregoing tests by applying the Westergaard method which makes it possible to calculate deformations of a concrete slab as a function of the modulus of reaction of the subgrade. In the case under consideration, the deformations were known and also the modulus of elasticity of the concrete. From this it was possible to determine the modulus of subgrade reaction. The value found was 1.4 kg/cm^3, which corresponds very nearly to the value as measured directly.

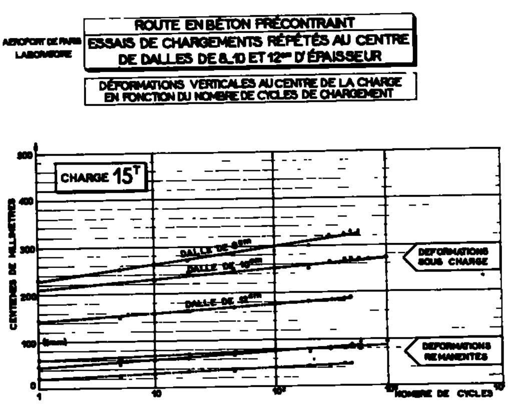

Figure 5. Orly test results: deflections under 15-ton load at center of slab after 10, 100, and 1,000 repetitions.

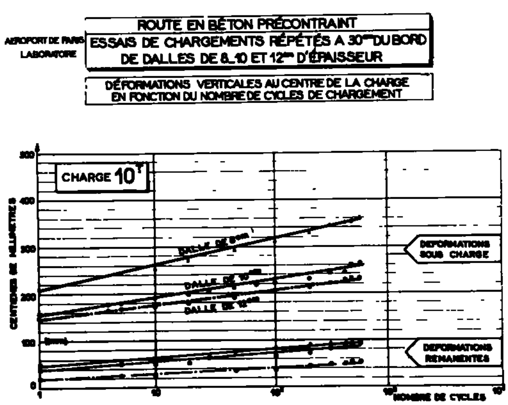

Figure 6. Orly test results: deflections under a 10-ton load on edge of slab and after removal of load.

This is of interest because it confirms the fact that the strength of the foundation ascertained before construction at the end of the rainy season had not appreciably improved during the course of the summer. The measurement of moisture content made by the laboratory in a hole drilled through the surface gave a similar result. It is interesting to check it with the usual formulas.

All the tests of which an account has been given were made on slabs 8, 10 and 12 cm in thickness without increased thickness at the edges. The possible reduction of the bearing capacity of a slab at its edges is a matter which was given attention and the Orly slabs had a thickness of 12 cm on one edge. Tests were made along this side at the very edge of the slab. They showed that the increased thickness made it possible to rely on a strength at the edge of the slab practically equal to that of a 12-cm slab regardless of the thickness at the center. It would, especially in the case of runways for light aircraft, make it possible to use the full bearing possibility at the center of a thin slab by simply increasing the thickness at the edges.

Effect of Variations in Temperature. —During the first tests the rate of longitudinal and transverse prestress was kept uniform at 10 kg/cm^2 (140 psi). The object of the following test was to determine the advantage of an anti-buckling device; that is, to see if, when a sudden rise in temperature is reflected by an increase in the stresses within the slab, a possible eccentricity of the flat jacks could cause one of the slabs to move upward in relation to the other. This test was carried out on August 22. In spite of the presence of various measuring devices, the results obtained were only of a qualitative nature. They have, however, unquestionably shown the necessity of anti-buckling devices. For one of the two joints in which only an extra thickness and an under-slab had been provided, inadmissible differences of level were ascertained between the two edges of the joint when jack pressure was applied. On the other hand, the other joint which had been provided with an anti-buckling device supported increases in pressure without the edges of the slabs showing a perceptible difference in level.

The test was effected by increasing the pressure in the jacks to their maximum. A pressure of 150 kg/cm^2 (2,100 psi) was thus reached, giving an average pressure in the slab of about 75 kg/cm^2. This corresponds to an increase of 65 kg/cm^2 as compared with the initial prestress rate; that is, a difference in temperature of the slab of more than 20 percent. This figure would amply correspond to diurnal temperature differences which may produce differences in level between the edges of the joints. Regarding the type of anti-buckling device to be recommended, the test simply showed that the system used at Maison-Blanche functions satisfactorily. Unfortunately, it is complicated to construct. A proposal permitting a considerable simplification has been put forward and has been tested. Tests showed that the device, although preferable to the simple increase of slab thickness at the joint, was not sufficiently effective to prevent vertical displacements measurable in millimeters. So far, only the device used in Algiers can be considered as completely effective.

Reduction of the Transverse Prestress. —The first tests were made with a uniform prestress of 10 kg/cm^2 in two directions at right angles to each other. The results were favorable and it was decided to reduce the transverse prestress by one-half, by relieving tension in alternate strands. A complete series of tests is to be made before winter. The first tests, made in the center of an 8-cm slab showed deformations under a weight of 25 tons similar to those observed under 30 tons with an average of twice as much transverse prestress (Fig. 7). At the edge the difference observed was some 10 percent in spite of the fact that the spacing between the tensioned strands was twice as great. It is clear that the tolerance of the slab thickness was only about 1 cm (½ cm at the top and ½ cm at the bottom). Displacement of the loading due to variations in the thickness of the slab may accordingly cause variations in the deformations of as much as 15 percent. The first tests performed after eliminating the tension in alternate strands, nevertheless, show that the influence of transverse prestress on the bearing strength is weak in the case of a slab that is not cracked and that a reduction of this transverse prestress by 50 percent reduces the bearing capacity only 10 to 20 percent. This fact was sufficiently arresting to warrant continuation of the tests by doing away with all transverse prestress in a certain area to see how the slab would behave.

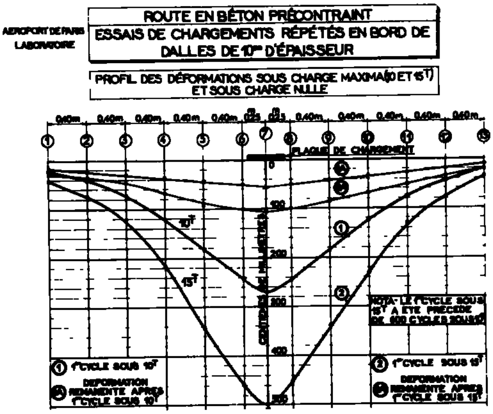

Figure 7. Orly test results: transverse deflections under load.

These tests were first made at various points in the 12-cm slab. It is interesting to compare results of the tests made with a 15-ton load applied at the center of the 12-cm slab with the tests made at the same point with the same weight but without transverse prestress. It was noted that the deformations under load as well as the residual deformations were very close to those observed with the transverse prestress, the difference being of the order of magnitude of test errors.

Other tests were performed with a 10-ton load applied at a distance of 30 cm from the edges with transverse prestresses of 10 kg, then reduced to 5 kg and finally completely eliminated. Here also, the deformations were similar. Residual deformations were clearly lower in the case of the 10-kg transverse prestress, however. These tests would tend to make one conclude that, in the case of roads, transverse prestressing is not essential. This conclusion is probably over-optimistic. Actually all the tests performed with a prestress of 10 kg/cm² (140 psi) were carried out within a few months after construction of the slab. The tests with reduced or with no prestress were made a year later, however. That is, the strength of the concrete, especially of concrete set under prestress and which therefore had no cracks, was greater in the second case than in the first. This point should be checked further.

Tests for the bending strength of the concrete were performed during construction. It would be of interest to prepare test specimens now and to determine the bending of strength of the concrete once more. These tests would make it possible to see to what extent the slight deformations under loading of the slab without transverse prestress may be due to an increase in the strength of the concrete.

Another point which the tests have not succeeded in elucidating is the extent to which the concrete near the longitudinal construction joint at the center line of the slab would react to the absence of transverse compression. To the present time tests have been performed at the center of the slabs or near the edges. Do the results of these tests apply to the area near the central joint and can it be deduced accordingly, that, in a slab that has at no time undergone transverse prestressing, the longitudinal prestressing would be sufficient to produce the results which have been noted?

Test Road at Fontenay-Trésigny

The interest in the techniques of prestressed concrete construction has led the French government to undertake construction of an experimental road on which various prestressed road systems may be tested.

The 2.5-km long by-pass of the Paris-Strasbourg road at Fontenay-Trésigny which will be subject to heavy traffic was selected for this purpose.

To benefit from developments in the field of prestressed concrete realized by various specialized engineering firms, the 2.5-km test road was divided into three sections, each to be constructed by a particularly qualified contractor.

The Société Campenon Bernard, which built the runway at Maison-Blanche, was given a section 1,140 m long (3,610 ft). The Société des Grands Travaux de Marseille was given a 350-m length (1,148 ft), the Société Boussiron 530 m (1,738 ft) and a group of three specialized concrete pavement contractors another 350-m section.

The systems selected by the various contractors were, for the most part, based on the over-all study of the project made by Chief Engineer Peltier, Director of the Central Laboratory of the Ponts and Chaussées, which was published in the October 1958 issue of the "Revue Générale des Routes."

The various proposed design systems correspond to the different possible types of prestressed concrete road construction; namely, external prestressing between two fixed abutments, external prestressing with elastic joints to insure even distribution of slab stresses and internal prestressing by cables.

The Government's purpose in proposing the construction of the test road was, in the first place, to investigate various possibilities and to study the largest possible number of different systems. The engineering companies entrusted with the project accordingly divided their sections into several parts jointed by joints, thus affording considerable scope for inventive skill.

The Société Campenon Bernard divided its section into three parts. The first part consisted of a slab prestressed by jacks between two fixed abutments 500 m (1,640 ft) apart (Fig. 8). The second part was also constructed between two fixed abutments but included various types of elastic joints. A last 150-m section was cable prestressed.

The abutments designed by this company consist of thick concrete slabs, the lower parts of which are toothed. The abutment functions through its weight on the one hand and on the other through the resistance furnished by the teeth which act as anchor spuds.

Abutments of this type can be constructed to any desired length and provide high thrust resistance.

In the 500-m (1,640-ft) section with elastic joints, the company proposed three types of joints: one with longitudinal cables in a sub-slab, another with spring joints of the "traction-compression" type composed of tensioned wires in metal tubes, and, a joint consisting of a trapezoidal slab section acting as a wedge between two adjacent slabs. The wedge is pressed against the slabs by springs and its movement between the slabs is facilitated by roller bearings. A beam between the two slabs balances the wedge.

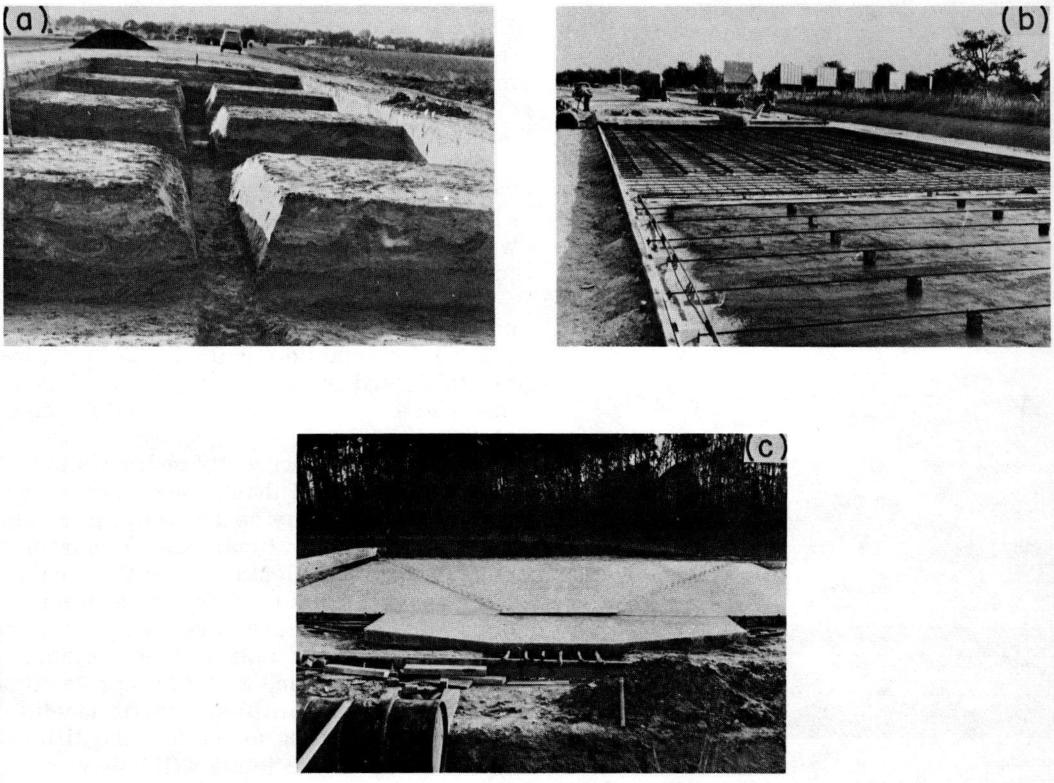

Figure 8. Fontenay-Trésigny Test Road: prestressing with jacks— (a) fixed abutments, foundation ready for pouring; (b) elastic joint before pouring slab; (c) trapeze-shaped elastic joint.

This last system would appear to be based on the first prestressed concrete runway designed by Freyssinet at Orly in 1947. The purpose of the joints was to maintain constant stresses in the slabs in spite of variations in the temperature, the stresses on the abutments being reduced proportionately. The end abutments for this section may be lighter than those for the first two sections. They are of the same type but are simply shorter. Pavement continuity is insured by metal tooth joints between slabs. The company provided transverse prestressing by strands. The slabs are 12 cm thick with additional thicknesses up to 18 cm under the edges and at the joints. Transverse abutments were constructed to absorb thrusts at the outer edge of curves.

The Société des Grands Travaux de Marseille divided its 350-m long section into three parts, each 117 m (383 ft) long. Cables were used for longitudinal and transverse prestressing in all cases (Fig. 9). Prestress values vary from one section to the other, however.

The main objective of this company was to study the problem of insuring pavement continuity between the various sections. Special joints consisting of steel elements and neoprene blocks furnished additional external prestressing.

The Boussiron Company planned two sections, 115 and 415 m (377 and 1,350 ft) in length, respectively. The first section was a 12-cm slab with crossed strands, the second, a mobile slab with elastic joints between two abutments of a new type designed by the company.

Three types of elastic joints were constructed: a spring-type joint with wires in tubes similar to those used by the Société Campenon Bernard; a snythetic rubber strip joint and, last, a pneumatic joint consisting of a rubber air chamber between two cast steel elements which function like pistons. Pavement continuity is insured by a metal tooth joint. The pavement is 12 cm thick as in the preceding cases.

Last, the group of specialized concrete pavement contractors is the only one in which transverse prestressing has been eliminated altogether. Concrete thickness was increased to 15 cm and a longitudinal joint was provided.

The foregoing are the systems under construction at Fontenay-Trésigny. The test road should be terminated before winter 1959, and traffic will be directed over the road as soon as construction is completed.

The test road presently under construction in France will thus provide considerable data which may be added to information obtained by observation of the Melsbroek runway and the Belgian test road. From the economic point of view, the systems being studied in France are certainly more costly than those constructed in Belgium which, from now on, would appear to offer competitive possibilities. It will nonetheless, be interesting to see how the different types of elastic joints will behave and to see if the toothed metal surface joints to maintain continuity of the pavement above the elastic joints will be as effective under traffic as the jack blocks used in the runways at Melsbroek and Algiers.

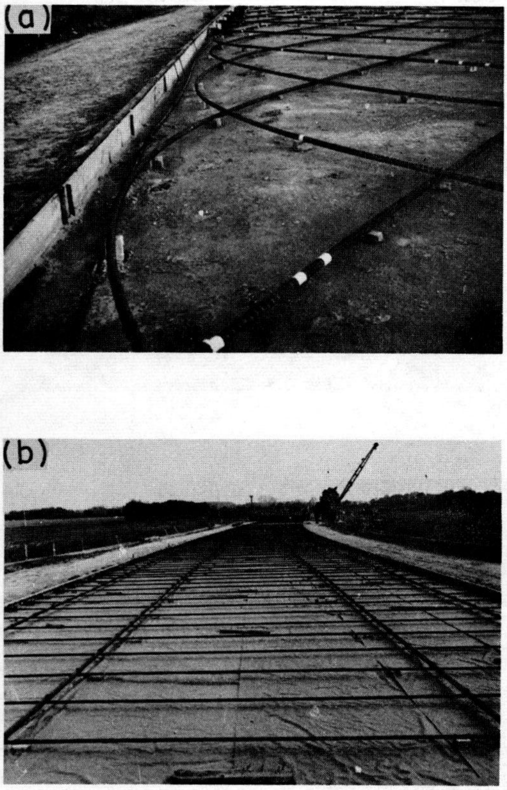

Figure 9. Fontenay-Trèsigny Test Road: prestressing with cables—(a) longitudinal and transverse cables (note small number of longitudinal cables); (b) cables at 45 degrees to axis of slab.

CONCLUSION

The tests which have just been described show that it is possible to construct prestressed concrete roads, at least on a straight section with a regular gradient, that it can be economical and that, from the point of view of the wearing qualities, the complete absence of joints will give it an unquestionable technical superiority. All the problems raised by this method of construction have certainly not been solved, however. For if airfield runways are always straight and practically horizontal, the same is by no means the case with roads. It is therefore necessary, when designing a road and considering the use of prestressed concrete, to divide the road into straight sections and curves or changes of gradient to be studied individually. The need for abutments at certain intervals, the possibility of constructing these abutments of ordinary or even prestressed concrete with cables anchored in the ground or not—as was demonstrated by the Swiss engineers in the case of a curve—all these problems can be solved in various ways. The credit of the engineers who tackle these problems will first be established by the very fact that they will have to solve them.

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Discussion

PHILLIP L. MELVILLE, Engineer, Airfields Branch, Office, Chief of Engineers; and PAUL F. CARLTON, Chief, Research Branch, Ohio River Division Laboratories—Periodic reviews of developments in the design and construction of prestressed concrete pavements are helpful to engineers engaged in this field. Mr. Mayer's paper is a timely report on the present state of the art in Europe. From his paper it is obvious that construction of prestressed pavements is progressing faster in Europe than in the United States.

It is unfortunate that definite correlations between pavements described by Mr. Mayer and current American practice cannot be established. Such data as are necessary to establish this correlation include: the modulus of rupture of the concrete, the modulus of subgrade reaction, and the configuration and magnitude of the vehicular or aircraft loading. In addition, the inclusion of some reference to the actual design concepts used would have added materially to the analysis of these European pavements.

Mr. Mayer makes two statements in his paper which the writers believe merit further attention. The first concerns the assumption that the transverse distribution of traffic will be such as to produce a uniform settlement of the pavement structure. It is believed questionable that such a condition will obtain, particularly with regard to airfield pavements. Statistical data, developed by the Corps of Engineers, on the lateral distribution of aircraft show that there is a heavy concentration of traffic along the central portion of such pavement features as taxiways and runway ends. This "channelization" of traffic is a function of the pavement width and the configuration of the aircraft landing gear. Obviously the effect of wheel loads, as related to producing settlement of the pavement structure, becomes more severe as the number of applications of the loading in a traffic lane of restricted width is increased. This has been verified for a number of airfield pavements included in the condition survey program of the Corps. Data on the deformation of both rigid and flexible airfield pavements under the effects of channelized traffic were reported to the Highway Research Board in 1959 by Sale and Foster.[1]

The second statement contained in Mr. Mayer's paper which the writers question concerns the significance attributed to repetitive plate loading tests. Although so-called static loading tests with a plate or an actual wheel will yield much worthwhile information as to pavement strains and deflections, the results cannot be translated fully into parameters of a pavement performance equation. Reliance on repetitive plate tests may lead to unconservative concepts relative to evaluating accurately the load-carrying capacity of pavement structures. Experience of the Corps of Engineers has shown that use of controlled traffic tests enables potential points of weakness to be observed more clearly. The advantages of controlled testing are threefold: stress reversals are produced at all points in the traffic area (especially important for prestressed pavements with their associated high negative moments), the effects of hygrothermal stresses are more fully evaluated, and localized areas of non-uniform subgrade support (if existent) may be detected.

It is also noted that European practice has been, almost without exception, to construct prestressed pavements on high-strength subgrades; that is, k values ranging from 200 to 500 lb per cu in. As a result, little attention is given in the paper as to the effect which subgrade strength has on the design requirements for prestressed pavements. Certainly it would be erroneous to develop the attitude that prestressed pavements are limited to high-strength subgrades only. On the other hand, data recently developed from accelerated traffic testing of two prestressed pavements constructed by the Ohio River Division Laboratories have indicated that while the utilization of low-strength subgrades without special treatment is feasible, the load-carrying capacity of the pavement is greatly influenced by the subgrade strength. For example, the average life of eight items in one of the ORDL test pavements was approximately 1,400 coverages of a 265,000-lb gear load on a 4-wheel twin-tandem wheel configuration having tire inflation pressures approximately 300 psi. This pavement was 9 in. thick and supported by a subgrade ranging in strength from 60 to 85 lb per cu in. The modulus of rupture of the concrete was 880 psi and the net prestress ranged from 200 to 400 psi. By contrast, the life of an almost identical prestressed pavement at Biggs Air Force Base, Texas, and supported on a subgrade having a modulus of 300 lb per cu in. is estimated to be more than 100,000 coverages under the same magnitude of loading.

With regard to the replacement of transverse prestressing with transverse reinforcing, the full-scale accelerated traffic tests referred to have shown that for airfield pavements constructed on low-strength subgrades, transverse prestressing is desirable. This appears to be particularly true where traffic is concentrated along a longitudinal construction joint. In the ORDL tests, the pavement life was increased substantially in test items containing transverse prestressing as compared to items with transverse reinforcing only.

A. MAYER, Closure—It was very gratifying for the author to read the constructive discussion of his paper by Mr. Melville and Mr. Carlton. He will try hereafter to answer their remarks.

The first question is on the existence of a definite correlation between European pavements and American current practice. This can be answered as follows: the concrete used in road construction is practically always a 350 kg/m^3 (6½ bags per

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[1]Foster, C.R., and Sale, J.P., "Accelerated Proof-Tests of Runway Pavement, Columbus Air Force Base, Mississippi." HRB Proc., 38:183-218 (1959).

cubic yard) concrete, placed with an $\frac{E}{C}$ factor of 0.4 to 0.45, that is, probably, somewhat dryer than is usual in the States. Compressive strength would be around 400 kg/cm^2 (6,000 psi)[2], flexural strength 60 kg/cm^2 (840 psi)[2] at 28 days.

The modulus of reaction of the subgrade which Messrs. Melville and Carlton would like to know, is something that differs considerably from one site to the other; this point will be discussed hereafter as an answer to their third question.

The loadings for which the different pavements were designed are:

1. For the roads in France, 13 metric tons axle load (that is, 6.5 tons per wheel), which is the French standard; in other European countries, 5 tons per wheel.
2. For airfields, the international standards are used. Melsbroek in Belgium, Kōln in Germany, Algiers in France are three Class A airfields designed to handle all types of commercial planes. The three NATO airfields of Wunstorff, Hopsten, and Nordholtz are tactical airfields designed to NATO standards (20,000 lb per wheel, 150 psi tire pressure).

The second question raised by Messrs. Melville and Carlton refers to the writer's assertion that, unlike tests where repetitive loads are constantly applied to the same point, actual pavement loads should be "uniformly distributed on the slab" so that there would be no deformation, but a uniform settlement. The writer agress that his broad statement does not cover all cases and that, especially on taxiways, the traffic is channelized within two lanes. He is quite ready to correct the sentence to "loads would actually be more evenly distributed on the slab." But inasmuch as the loaded area would be broader, the end result would be that, at least to some extent, the local deformation would be less.

Messrs. Melville and Carlton mention that traffic tests give more reliable values than repetitive plate loading tests; they therefore question the value of the Orly experiments. The author would remind them that when he visited the testing station of the Ohio River Division late in 1957, a prestressed slab had been completed several months before and the station was awaiting allotment of the necessary funds before the traffic tests could be started. Such things also happen in Europe, the only difference being that in the United States there may be some hope of getting enough money to carry out traffic tests, whereas at Orly an attempt had to be made to solve the problem without going to such expense. With the setup as it was at Orly 800 loadings could be achieved in a day; this cut down the cost of the tests considerably. It certainly requires using a larger margin of safety, as the applied loads were 10 and 15 metric tons on the edge and in the center of the slabs, as compared to a maximum wheel load of 6.5 tons.

The results of the Orly tests yield worthwhile information but the author quite agrees that these tests must be interpreted before the results are used in actual design.

The following point mentioned in the discussion is the fact that European prestressed concrete pavements have almost without exception been built on high-strength subgrades. This has long been true; it is also true that in so doing European engineers did not take into account the main advantage of prestressed structures, their flexibility and their ability to distribute applied loads. Actually the Orly tests were largely intended to show the possibility of laying prestressed pavements on practically any base, as long as it was strong enought to stand the movement of the construction equipment. The slabs were poured on 6 in. of non-compacted sand and gravel over a sandy silt saturated subgrade with a K value of 1.4 kg/cm^3 (50 pci), which is about as low a value as can be expected anywhere.

But the Orly slabs were test slabs, not actual parts of a final structure. For these structures, the engineers did not want to take any risk, so they largely overdesigned the base. This was certainly the fact for the first Algiers runway, as well as for the Bingen test road. It is not true for some recent pavements, such as the most recent runways built in Germany and Belgium, where the base was restricted to what was

[2]Measured on cubic samples, which accounts for a difference of ¼ as compared to U.S. results.

actually necessary to prevent frost action. As mentioned in the paper, a reduction in cost of 5 DM ($1) per square meter of base was taken into account in the comparison between prestressed and ordinary concrete for the German airfields. The discussors' remark is important, but the author sincerely believes that it only applies to the first runways built. It is not conceivable that the engineers in charge would not take full advantage of the fact that prestressed concrete, due to the absence of joints or cracks, can distribute the load much better than ordinary concrete, in reducing the strength of the base and thereby reducing its cost. The information Messrs. Melville and Carlton obtained from traffic tests on a 9-in. pavement with a 265,000-lb gear load on a 60- to 85-pci modulus of reaction subbase, is interesting; but would anyone in actual construction build a 9-in. thick runway on a 60- to 85-pci subbase, without placing at least 6 to 9 in. of sand and gravel so that equipment can be used without completely remolding the soil.

Concerning the last point, the author was glad to hear that the full-scale accelerated traffic tests had proved in favor of transverse prestressing. Such has also been the conclusion in Europe for runways and taxiways. For runways, transverse prestressing has proved to be no more expensive than plain reinforcing, and certainly more effective. The only point now under investigation is the question of roads. In Switzerland, road pavements less than 15 ft wide have been constructed without transverse reinforcing or prestressing; the same thing has been done on the test road at Fontenay-Tresigny on one of the sections, but there the slab has been thickened from 5 in. to 6 in. It is believed to be better in all cases to provide for some transverse prestressing, even for roads, with only one strand every 4 ft.

The Case for Skewed Joints

R. H. COOLEY, Statewide Paving Engineer, Portland Cement Association, Los Angeles District

The use of skewed joints in concrete pavements in California dates back to 1932, when an experimental installation was built on U. S. 40 southwest of Sacramento.

No further installations were made until 1951. Influenced at least in part by the satisfactory performance of the 1932 installations, skewed joints have since been used on portions of at least eight projects. In these, various joint spacings and angles of skew were used and their performance observed.

The results of these studies indicated that: (a) when measured by the number and magnitude of faulted joints there was a decided advantage in favor of the skewed joints; (b) the joints spaced at 15 ft and 20 ft showed a marked superiority over joints spaced at larger intervals; and (c) after study of heavy truck specifications for axle and wheel spacing, a skew of 2 ft in a 12-ft slab with the obtuse angle at the outside edge of the forward slab was adopted.

A description of various projects and a summary of performance of various joint spacings is included.

●THE IDEA of placing transverse joints at other than right angles to the pavement centerline is not new. A review of the literature (1) shows that patents were issued for skewed joint systems in 1906 and later in 1918. Drawings for the 1906 patent show a skew of 5 ft for a 12-ft lane width. The drawings do not show whether the obtuse angle formed by the skewed joint and the outside pavement edge is ahead of or behind the joint in the direction of travel.

The 1918 patent was issued for a joint with a 45-deg skew. However, this skew does not extend across the full pavement width. Instead, the joint is at right angles to pavement centerline for a distance of about 18 in. in from each pavement edge.

Skewed transverse joints conforming to these patents, or to other skew designs, were installed in experimental concrete pavements and were occasionally used in normal construction work during the 20-yr period from 1910 to 1930. Very little is known about the performance of the skewed joints constructed during this period.

SKEWED JOINTS IN CALIFORNIA

The modern day development of skewed joints started in 1932. In that year the California Division of Highways installed skewed joints on a section of a two-lane highway located about 6 mi southwest of Sacramento on US 40. The joints were hand formed, with a skew 4 ft in the 10-ft wide traffic lanes. This section is now located on the westbound lanes of a four-lane divided highway carrying heavy traffic between Sacramento and San Francisco. The 1957 traffic count for this route was more than 21,000 vehicles per day of which about 17 percent were trucks. Figures 1 and 2 show the present condition of these skewed joints after 26 yr of service. The joints are not

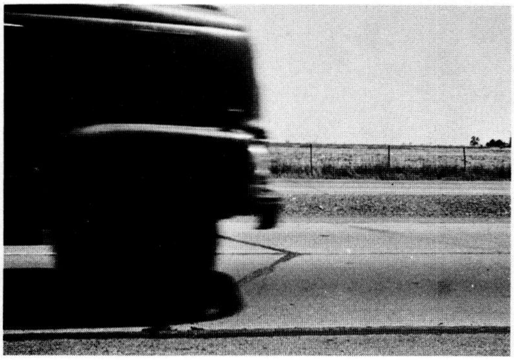

Figure 1.

Figure 2.

faulted and a 1958 inspection showed no structural distress in the form of corner breaks. The absence of corner breaks is particularly significant, because the direction of skew forms an acute angle at the outside corner ahead of the joints in the direction of travel. These acute angle corners are critical locations for corner breaks because the outside wheel path of heavy trucks is close to outside edges of the 10-ft wide lanes. The absence of corner breaks indicates that the skewed joints have the effect of providing a high degree of load transfer. As a result stresses in the acute angle corners are kept within safe limits.

Further construction of skewed joints in California was deferred until service performance of the 1932 project could be established. World War II caused additional delays, and it was not until 1951 that construction of projects with skewed joints was resumed.

An important item in the 1951 construction program was an experimental jointing project designed to compare (a) the relative performance of skewed and right angle joints and (b) the relative performance of joint spacings ranging from 15 to 60 ft. This 7,000-ft project is located on US 99 between Los Angeles and Bakersfield in the Tejon Pass of the Tehachapi Mountains. The 8-in. plain pavement is 24 ft wide, constructed lane-at-a-time with a keyed longitudinal joint tied with $\frac{5}{8}$-in. bolts at 30-in. centers. The pavement was placed on a cement-treated subbase 4 in. thick and 24 ft wide.

No expansion joints were used. Both right angle joints and joints with a skew of 3 ft in 12 ft were constructed in various sections of the pavement at spacings of 15, 20, 30, 40 and 60 ft. The contraction joints were formed by 2-in. asphalt impregnated fiber strips. Some difficulty was encountered in holding the strips vertical and flush with the pavement surface. As a result there has been considerable spalling of the joints. Figure 3 shows a skewed joint in the section with a joint spacing of 15 ft. Unsatisfactory joint alignment and spalling are evident.

The experimental sections are on the two southbound lanes of a four-lane divided highway which carries more than 12,000 vehicles per day, including 3,200 trucks. Most of the southbound trucks are fully loaded with cargoes bound for Los Angeles. As a result the experimental joint sections have been subjected to a severe test.

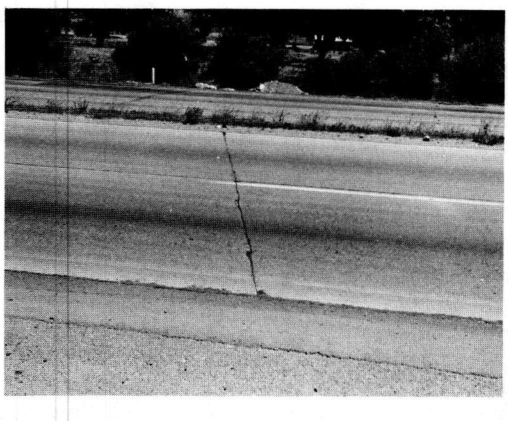

Figure 3.

Truck speeds over the section vary with the grades which are not particularly steep in this section of the Tejon Pass. Most southbound trucks stop at a loadometer station shortly before entering the section. The speed of several 5-axle truck-trailer combinations, when checked, showed an average speed of 21 mph over part of the test section. At one check point where grades were slightly steeper the average speed was 16 mph and at another location the trucks averaged 40 mph. Truck traffic over this route is almost as heavy at night as during daylight hours.

The flexural strengths of the concrete on this project were on the low side. As a result there is considerable joint faulting and distress in the form of corner breaks. For experimental purposes the low strengths and resultant defects have served to accelerate the effects of traffic on the various joint designs, permitting earlier comparison of test results.

A comprehensive survey of this experimental pavement was made in 1955. The results of this survey are given in Tables 1 and 2. Table 1 gives the relative performance of skewed and right angle joints. The data shows marked superiority of skewed joints over the right angle joints. Skewed joints had 90 percent fewer corner breaks than right angle joints, and skewed joints had 75 percent fewer faults than right angle joints.

Table 2 gives the relative performance of the various joint spacings. These data show that both corner breaks and faulted joints increased as the joint spacing was increased from 15 to 60 ft with marked superiority for the 15-ft joint spacing. Data from both tables show that skewed joints at a spacing of 15 ft gave the best performance of any combination tested.

This project was inspected again during the fall of 1957. At that time the pavement had begun to show considerable distress, particularly in sections where the longer joint spacings had been used. However, the 15- and 20-ft sections with skewed joints were still in remarkably good condition when compared to right angle joints at the same spacings.

Since 1950 skewed joints have also been installed in portions of seven additional projects. Except for one project constructed in 1957, the skew was either 2.5 or 3.0 ft in 12 ft. On the 1957 project the skew was reduced to 2 ft in 12 ft.

This 2-ft skew in 12 ft was adopted after considerable study of specifications for truck axles and wheel spacings. Figure 4 shows the tire imprints of a tandem-axle, dual-wheel truck equipped with eleven 22.5 tires in relation to a skewed joint. The dual wheels are spaced at 12¾ in. center to center and the distance center to center of the inside wheels on each axle is 59½ in. The tandem-axle spacing is 52 in. The skew of 2 ft in 12 ft is the minimum amount which will result in movement of the dual wheels across the joint one-at-a-time.

TABLE 1

Joints: Type	Joints: Panels (no.)	Corner Breaks (%)	Faulted Joints (%)
Skewed	133	2	11
Right angle	158	22	76

TABLE 2

Joints: Spacing (ft)	Joints: Panels (no.)	Corner Breaks (%)	Faulted Joints (%)
15	135	2	9
20	62	12	72
30	43	25	78
40	31	30	80
60	20	40	95

It will also be noted that the joint is skewed in a direction that results in an obtuse angle at the outside edge of the forward slab. This affords additional protection against corner breaks at this critical location. It is true, of course, that there is an acute angle at the inside corner ahead of the joint. However, this inside corner is less susceptible to corner breaks because it is tied to the adjacent slab on the other side of the longitudinal joint.

Until recently all California pavements have been paved lane-at-a-time. After the end of World War II it became general practice to saw transverse joints. The combination of lane-at-a-time paving and joint sawing resulted in certain complications where skewed joints were used. No particular problems were encountered in sawing the first lane placed. However, it was found that subsequent lanes had to be sawed at just the right time to prevent transverse cracks from forming ahead of the saw cut. If sawing was just a little too late transverse cracks had already formed, and these cracks were at about right angles to pavement centerline rather than skewed. Even when the second slab placed had not cracked, but enough tensile stress had developed to cause tension cracking ahead of the saw, this crack did not follow the direction of skew but formed a right angle to the outside edge or longitudinal joint.

Figure 5 shows a procedure for avoiding these difficulties where transverse joints are sawed in lane-at-a-time construction of four-lane highways. It is necessary to construct the outside lane first and saw the skewed joints. When the adjacent inner lane is placed the joints can be sawed at right angles to pavement centerline. With this arrangement the skewed joints are in the outside lane which carries almost all of the heavy truck traffic. It should be noted that this procedure is not necessary where joints are formed in the plastic concrete, or where the pavement is placed full width. In these cases the skewed joints can be continued across the full pavement width in a chevron pattern where joints are formed in the plastic concrete, or in a straight line continuation of the skew where joints are sawed.

Skewed joints were placed on a section of the Hollywood Freeway constructed in 1951. This pavement has the same design as the experimental project—an 8-in. plain pavement on a 4-in. cement-treated subbase. This section of the Freeway

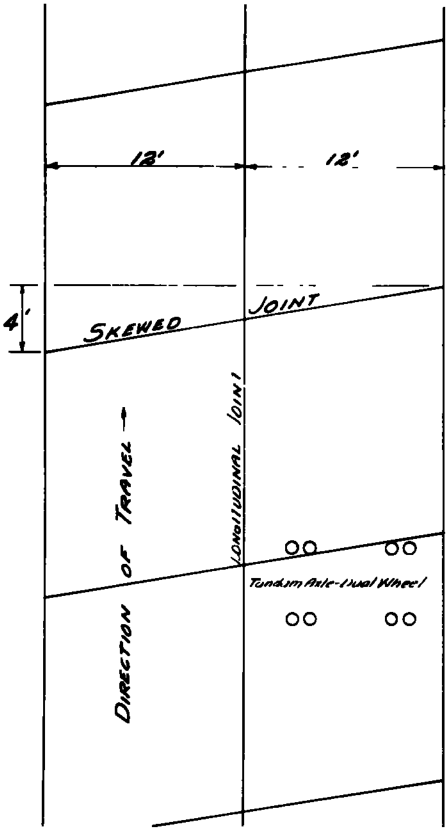

Figure 4. Full width (24 ft) construction.

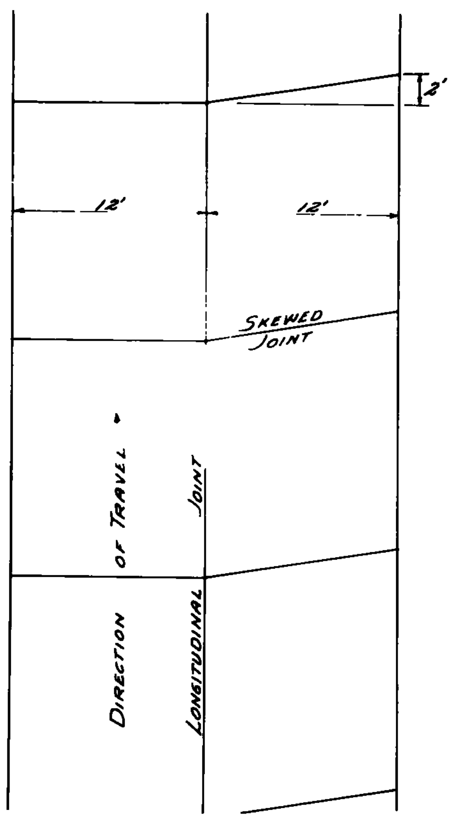

Figure 5. Lane at a time construction.

carries 190,000 vehicles per day with 6.6 percent, or 12,500 trucks per day. Present condition of the skewed joints is shown in Figure 6. They are free of faults and there are no corner breaks.

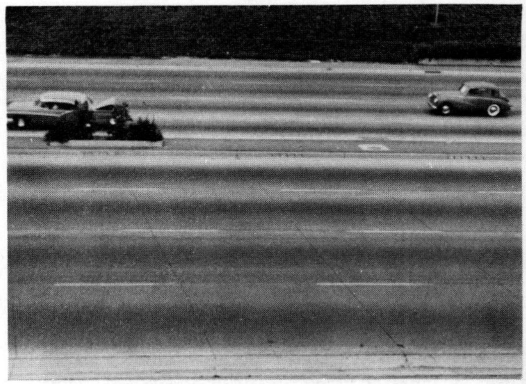

Figure 6.

In 1952 skewed joints were also installed on a section of the San Bernardino Freeway, east of Los Angeles. Again the design is the same, an 8-in. plain concrete pavement on a 4-in. cement-treated subbase. This route carries about 90,000 vehicles per day, including a large volume of heavy truck traffic. Figure 7 shows the present condition of the skewed joints. Even though the direction of skew makes an outside acute angle ahead of the joint in the direction of travel, there are no corner breaks or other evidence of structural distress, and there are no faulted joints.

SKEWED JOINTS IN WASHINGTON

In 1954 the Washington State Department of Highways installed sections with skewed joints on several four-lane projects located on US 99. This is the principal route between Portland, Oregon and Seattle, Washington. It is now Interstate Route 5, and carries a high volume of heavy truck traffic.

These projects have plain concrete pavements, 8 in. thick, placed on a granular subbase 6 in. or more thick. Expansion joints were placed only at bridge ends and other fixed structures. Transverse contraction joints were spaced at 15 ft with a skew of 2 ft 8 in. in 12 ft. The pavements were constructed lane-at-a-time with a tied longitudinal center joint. The skewed joints have been placed in two patterns: (a) a chevron pattern (Fig. 8) and (b) continuation of the skew in the same direction across 2 lanes (Fig. 9).

The exceptionally good riding qualities and the excellent performance of the experimental sections led to the adoption of skewed joints as standard for all projects with joints constructed while the concrete is plastic. Where sawed joints are cut in hardened concrete the joints are at right angles to pavement centerline.

CONCLUSIONS

1. Skewed transverse contraction joints spaced at 15 ft have been in service for 26 yr under heavy truck traffic. Performance of the skewed joints is excellent. They

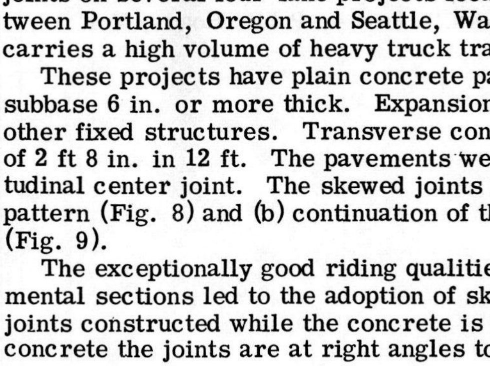

Figure 7.

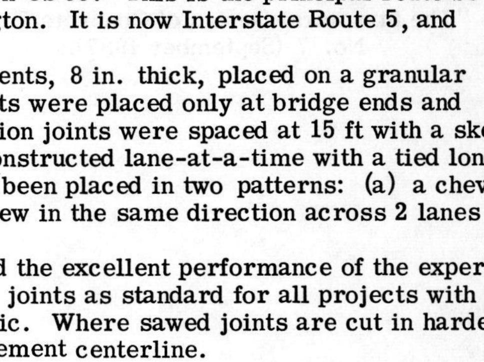

Figure 8.

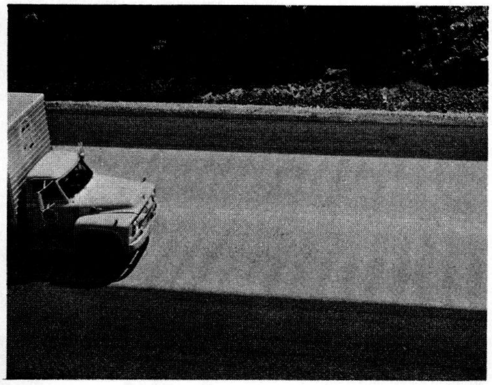

Figure 9.

are free of faults and the pavement is free of structural distress.

2. An experimental jointing project was constructed in 1951 on a route carrying large volumes of heavily loaded trucks. Skewed and right angle joints were installed at spacings of 15, 20, 30, 40 and 60 ft. As given in Tables 1 and 2, the skewed joints have performed much better than right angle joints and short joint spacings have proven superior to longer joint spacings. The best performance was on skewed joints spaced at 15 ft.

3. A 1957 study of truck axle and wheel spacings resulted in adoption of a skew of 2 ft in 12 ft, with the obtuse angle at the outside edge of the forward slab.

4. The performance of California and Washington pavements indicates that moving wheel loads across transverse joints one at a time by means of skewed joints, results in improved riding qualities and better load transfer across the joints.

REFERENCE

1. "The Structural Design of Concrete Pavements." Part 4, Public Roads, Vol. 17, No. 7 (September 1937).

An Experiment in Pavement Slab Design

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A number of experimental features were introduced in a portland cement concrete pavement on a regularly scheduled construction project by the Illinois Division of Highways in 1952. Included in the experimental portion of the project were sections of nonreinforced pavement with and without transverse joints, and reinforced sections having the Illinois distributed steel reinforcement with a variety of transverse joints and joint treatments. Although the pavement is still in excellent condition, certain early behavior trends are now indicated.

● THE EXPERIMENTATION with which this report is concerned involves the introduction and subsequent study of the behavior of several experimental features in a portland cement concrete pavement constructed on a regularly scheduled paving contract in 1952.

A major motivating factor behind the early planning of this research study was the uncertainty at that time as to the continued availability of steel reinforcement for concrete pavement because of the demands of warfare in Korea. Distributed steel reinforcement had been used in Illinois pavement since the late 1930's, except for a period during World War II when it was not available. Pavement placed during the war-time years without steel and with expansion joints at close intervals had not proved satisfactory in Illinois experience. The experimentation was therefore directed in a large measure toward a study of design types suitable for development as substitutes for the standard reinforced pavement. Other items of research include several types of transverse contraction joints under study to determine service behavior with respect to riding quality, durability and retention of seal; and two types of joint-sealing compound being evaluated qualitatively.

The construction project selected for the study is 4.3 mi long and located on US 66 immediately south of Springfield. It is identified officially as Section 110X-5, SBI Route 126, Federal Project FI-166(18), Sangamon County. US 66 is one of the heaviest traveled rural pavements in the state and at the location of the test project carries presently about 12,600 vehicles per day, including 2,200 trucks and buses. The test pavement consists of two 12-ft lanes carrying southbound traffic. It is separated from the northbound pavement by a grassed median 30 ft wide.

The topography of the area of the test pavement is generally flat and the pavement lies on a slight embankment throughout most of its length. Soils are fine grained and reasonably uniform, classifying mostly as A-6 and in the higher plasticity range of the A-4 group.

The major construction operations on the test project included the placing of the embankment, a 6-in. trenched granular subbase, and the 24-ft width of experimental pavement. No special effort was made to obtain particular uniformity and precision of construction other than to observe the usual construction and inspection techniques of the Illinois Division of Highways.

The test pavement is a part of a Federal-participating construction project and the experimentation has been conducted in cooperation with the U. S. Department of Commerce, Bureau of Public Roads.

RESEARCH PURPOSES

The following research purposes were established for the study:

1. To make observations and comparisons of the behavior of the 1951 standard Illinois design of concrete pavement having a 10-in. thickness, welded-wire fabric reinforcement, and hand-edged full-depth metal-plate contraction joints at 100-ft intervals versus certain types of plain (nonreinforced) concrete pavement. The following sections of plain pavement were included for this purpose:
   a. A 10-in. uniform thickness without joints;
   b. A 9-in. uniform thickness without joints;
   c. A 10-in. uniform thickness with dummy-groove plane-of-weakness contraction joints at 20-ft intervals (no load-transfer devices); and
   d. A 9-in. uniform thickness with dummy-groove plane-of-weakness contraction joints at 20-ft intervals (no load-transfer devices).
2. To make comparative observations of the behavior of the Illinois standard full-depth metal-plate contraction joint and plane-of-weakness joints of the following basic types:
   a. Dummy-groove (with load-transfer devices); and
   b. Sawed (with load-transfer devices).
3. To make comparative studies of the riding quality and durability of full-depth metal-plate contraction joints having hand-tool finishing of joint edges, versus the same type of joint without hand-tool finishing.
4. To make comparisons of the effectiveness of a hot-poured rubber-asphalt joint sealer versus a cold-applied rubber-asphalt ready-mixed sealing compound.
5. To evaluate the effect of adhesion between sealing compound and concrete that might result from abrading opposing vertical faces of edged contraction joints.
6. To evaluate the comparative behavior of wetted-burlap cure and impermeable paper cure in the control of cracking prior to sawing transverse joints.

A statewide change in construction practice with respect to transverse contraction joints while the experimental construction work was in progress resulted in the control section being divided into two parts. One part was constructed with hand edging used at the transverse metal-plate joints as was the practice at the time the project was initiated. Hand edging was omitted on the other portion in accordance with the new practice that went into effect during construction.

To accomplish these research purposes, 11 test sections inclusive of the two control sections varying in length from about 1,000 ft to about 3,600 ft were established. They were constructed consecutively, the pavement being placed in a five-week period during May and June 1952.

PROJECT DESIGN

The general layout of the construction project is shown in Figure 1. Test sections, as previously mentioned, are built to serve as the two southbound lanes of the dual highway. Out-to-out width of these two lanes is 24 ft. They are on new location lying west of and parallel to the formerly existing 20-ft portland cement concrete pavement built in 1932 to serve traffic in both directions. Under a 1952 construction contract the old pavement was widened with a 4-ft strip of plain portland cement concrete placed on the west side, and the full 24-ft width was surfaced with 3 in. of I-11 bituminous concrete. The new surfacing serves northbound traffic.

A typical cross-section of the portion of the roadway occupied by the experimental pavement is shown in Figure 2. Shoulders are of turf, 6 ft wide between pavements and 10 ft wide outside of the pavements. Frontage roads are provided along each side of the throughway.

LOCATION AND LAYOUT OF TEST SECTIONS

It has been indicated previously that the experimental research features are included as design components of the southbound lanes of a dual highway. Principal

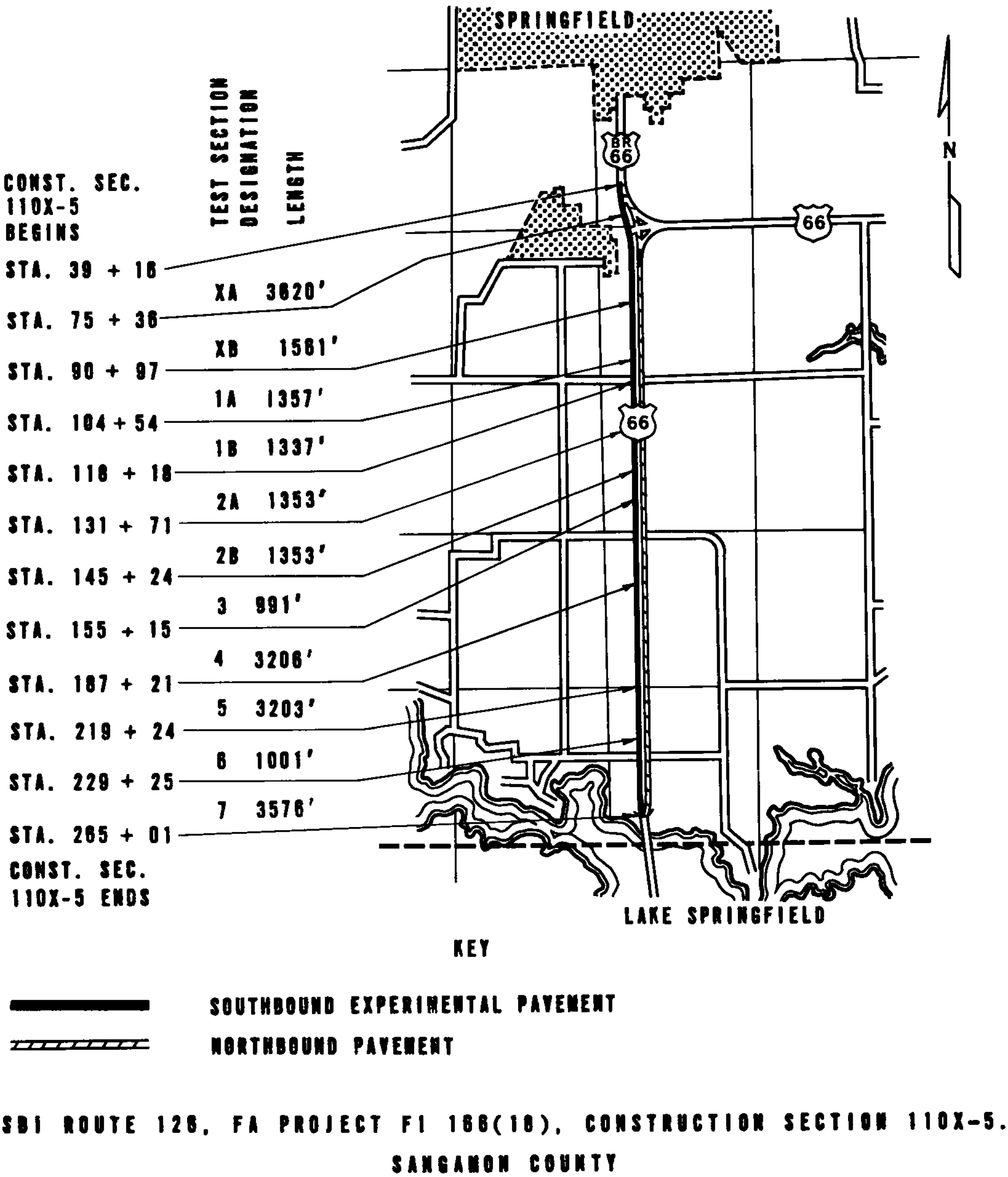


Figure 1. Location and layout of experimental sections.

design features and other pertinent data regarding the test pavements are given in Table 1.

Several features of design are common to all test sections. Some of these common features are apparent in Table 1 but others are not. Those not otherwise shown are as follows:

1. All pavement is uniform 24-ft width portland cement concrete.
2. A trench-type, undrained, granular subbase underlies all test sections. The

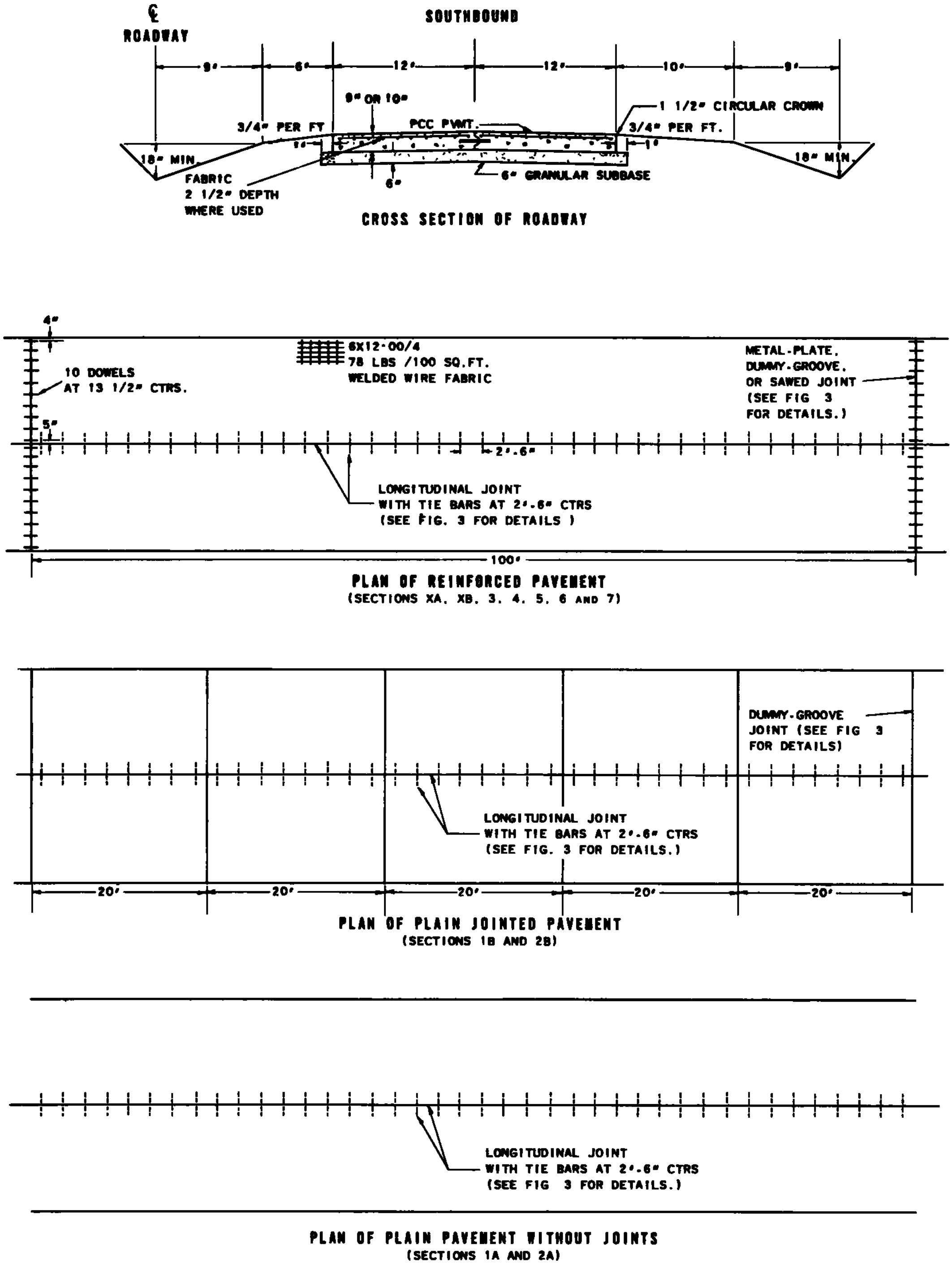


Figure 2. Typical cross-section of roadway, and plan sections of portland cement concrete pavement.

TABLE 1

PRINCIPAL DESIGN FEATURES OF TEST SECTIONS

Test Section	Station to Station	Length (ft)	Pavement Thickness (in.)	Reinforcement (lbs per 100-sq ft)	Panel Length (ft)	Joint Type	Load Transfer Devices
XA	39+16 to 75+36	3620	10	78	100	Metal plate (unedged)	Dowels
XB	75+36 to 90+97	1561	10	78	100	Metal plate (edged)	Dowels
1A	90+97 to 104+54	1357	10	None	-	Construction only	None
1B	104+54 to 118+18	1337	10	None	20	Dummy-groove	None
2A	118+18 to 131+71	1353	9	None	-	Construction only	None
2B	131+71 to 145+24	1353	9	None	20	Dummy-groove	None
3	145+24 to 155+15	991	10	78	100	Sawed	Dowels
4	155+15 to 187+21	3206	10	78	100	Dummy-groove (abraded faces)	Dowels
5	187+21 to 219+24	3203	10	78	100	Dummy-groove	Dowels
6	219+24 to 229+25	1001	10	78	100	Sawed	Dowels
7	229+25 to 265+01	3576	10	78	100	Metal plate (edged; abraded faces)	Dowels

Note: All transverse joints sealed with cold-applied rubber-asphalt compound except 18 joints of Section 7 sealed with hot-poured rubber-asphalt compound granular subbase has a design thickness of 6 in., and is 26 ft wide, extending 1 ft beyond the edges of the pavement on each side.

3. Longitudinal joint assemblies are of the metal-plate type with No. 5 deformed tie bars 2 ft 6 in. long at 2-ft 6-in. centers.

The design features in standard use in Illinois during the period of planning this experimental research are represented throughout several of the test sections so that the behavior of each experimental feature of the study that is a departure from the then-current Illinois standard may be compared, not only with that of other experimental types, but also with the behavior of the respective standard. Essential features of Illinois portland cement concrete pavement design in use at that time (1951) were previously discussed. None of the experimental features of this research except possibly the spacing of sawed contraction joints at 100-ft intervals, were innovations. However, none had been used previously in Illinois under the heavy traffic conditions anticipated for this pavement.

Experimentation Included in the Test Sections

Following are brief discussions of the study items that are departures from 1951 Illinois standard practice and which are the principal components of the present research.

Jointless Plain Concrete.—Test Sections 1A and 2A, 1,357 and 1,353 ft long, respectively, are plain concrete pavements without transverse joints. This basic type was included in the study because of existing evidence that similar pavements had rendered long and satisfactory service on many early-constructed miles of road in Illinois. It was considered important to investigate behavior of this type of pavement under heavier traffic densities and axle loads. Evaluation of its performance in comparison with that of other pavement types was contemplated as a possible means of developing a standby nonreinforced type of design suitable at least for periods of steel shortage.

Short-Panel Plain Concrete (without load-transfer devices).—Test Sections 1B and 2B, having dummy-groove contraction joints, are of a design not previously used on regular construction projects in Illinois, though sometimes used by other highway agencies. These sections are 1,337 and 1,353 ft long, respectively. The 20-ft panel lengths are without load-transfer devices. The sections are departures in these two respects from the main features of most of the other test sections where the standard panel length is 100 ft and dowel assemblies are present at contraction joints. This basic short-panel-length design, without steel for reinforcement or load transfer, is a second possibility being investigated for use during periods of steel shortage.

Sawed Contraction Joints.—Test Sections 3 and 6, 991 and 1,001 ft long, are of the Illinois standard design of 1951 except that the transverse joints were formed by sawing the hardened concrete. These two sections are similar except for the methods of curing. Section 3 was cured by the use of impermeable paper, whereas Section 6 was cured by the use of wetted burlap.

Dummy-Groove Contraction Joints.—Test Sections 4 and 5, 3,206 and 3,203 ft long, employ dummy-groove transverse joints in place of the metal-plate joints, but otherwise conform to the 1951 standard Illinois design. Section 4 differs from Section 5 in that an effort was made to abrade the edged faces of the joints of Section 4 prior to sealing.

Rubber-Asphalt Joint Sealing Compounds.—Test Section 7 varies from 1951 standard Illinois design only in the final treatment of the metal-plate contraction joints included within its 3,576-ft length. All of the contraction joints received abrading treatment, and 19 of the joints, including one nonabraded construction joint, were sealed with a hot-applied rubber-asphalt compound, whereas the remaining 17 contraction joints were sealed with a ready-mixed, cold-applied rubber-asphalt compound. Contrary to other test features, provision was not made for comparative evaluation between the rubber-asphalt compounds and the Illinois standard asphalt filler of PAF grades.

Abrading Opposing Vertical Faces of Contraction Joints.—Tendencies have been observed in the past in Illinois for loss of seal at joints to result not only from rupture of the sealing material and loss of adhesion between concrete and the sealing material, but also from separation between thin exterior films of mortar and the main body of the concrete, with bond apparently preserved between the contact face of the mortar film and the sealing material. Removal of the mortar film was the exploratory objective of the abrading process with improvement of adhesion as the ultimate objective. Dummy-groove joints of Section 4 and metal-plate joints of Section 7 were abraded.

Unedged Joints.—The section of pavement between Stations 39+16 and 90+97 and originally identified as Section X was to be established as a control section and was not intended to include experimental features when the research was initially planned. However, a statewide change in construction procedure, which eliminated hand edging of the metal-plate transverse joints in an attempt to improve riding quality, resulted in a change of plans for this section. It was decided that this section would afford an opportunity to compare the service behavior of edged and unedged metal-plate joints, and this purpose was accordingly added to the research program. Sixteen metal-plate contraction joints included within the section limits were finished with a hand edger in conformity with the standard practice earlier prevailing. The remaining 36 metal-plate joints were left unedged in accordance with the change in Illinois standard adopted about the same time as the research of Section 110X-5 got under way. The section having unedged joints is now identified as Section XA, and the section with the originally planned edged joints is identified as Section XB.

Pavement details of the individual test sections are shown in Figures 2 and 3. The contents of these figures require no special explanation.

SOILS

The test project traverses an area of dark-colored prairie soil where the undisturbed terrain is nearly level to gently sloping. Soil of the area is developed from thick to moderately thick loess and is medium-textured with moderately permeable subsoil. The preconstruction soil survey showed subsoils ranging from silty clay loam to silty clay over a layer of silt, the latter at depths varying from 4½ to 6 ft below the ground surface. The depth of borrow areas on the northern end of the project was limited to 4½ ft due to the silty nature of soil below that horizon. Soils are of A-4 and A-6 group classifications and are generally considered to be susceptible to pumping. A more detailed discussion of the subgrade soils is presented later.

TRAFFIC

US 66 is the main highway between Chicago and St. Louis and its traffic is one of the highest volumes of any rural

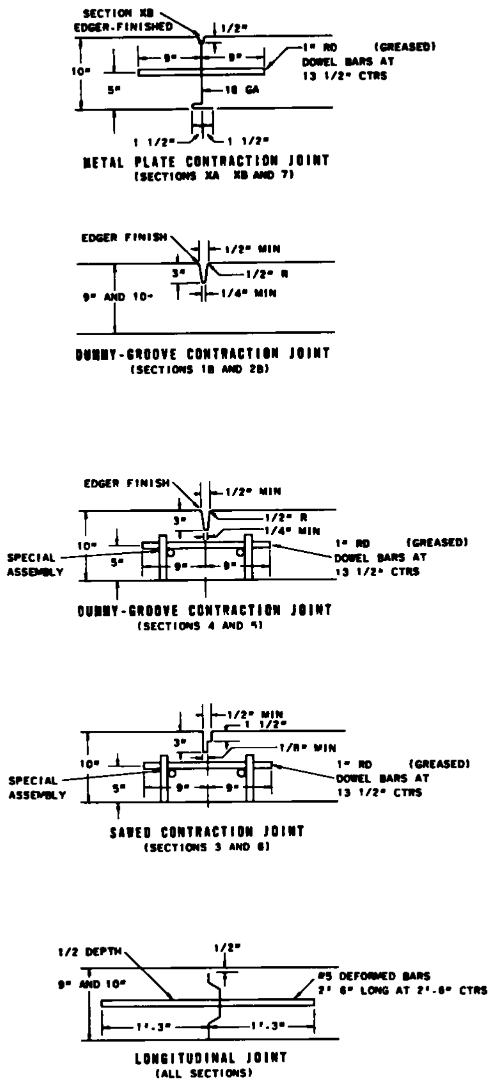


Figure 3. Details of contraction joints and longitudinal joint.

road in Illinois. Axle loads due to truck flow are likewise in the higher ranges of intercity routes. Traffic surveys in 1950 showed that the annual average daily volume carried by the two-lane pavement, at the location of Section 110X-5, amounted to 9,100 vehicles, of which 1,750 were trucks or other commercial types. The latter figure includes 805 tractor-truck semitrailer and full-trailer combinations. Traffic surveys made in 1953, about 1 yr after completion of the test project, showed that US 66, between the south junction of Springfield City US 66 and the Lake Springfield Bridge, now a divided, four-lane highway, served an average daily volume of 9,750 vehicles of all types, including 2,010 commercial vehicles with a further breakdown of 1,080 combinations. A survey of 1959 traffic shows that annual average daily volumes had risen to 12,600 total vehicles inclusive of 2,200 commercial types, including 1,400 combinations. The division between northbound and southbound traffic is approximately equal.

A permanent truck-weight station is located on US 66 approximately 35 mi south of the test project. It is believed that weight data recorded at this station are reasonably representative of heavy hauling over the test project. Although traffic volumes are considerably higher at the location of the test project than they are in the vicinity of the truck-weight station, classified counts at the two locations show that the heavier vehicles generally comprise through truck movements. It is therefore probable that

TABLE 2

TRUCK AXLE-WEIGHT AND VOLUME DATA

Weight Groups (pounds)	Daily Axle Loadings at Weight Station 35 Miles South of Experimental Project 1/			
	1953 Annual Average		1957 Summer Average	
	(number)	(percent)	(number)	(percent)
	Truck Single Axles			
Under 8,000	2,720	58	2,830	59
8,000 - 11,999	440	10	750	15
12,000 - 14,999	540	12	440	9
15,000 - 17,999	900	19	730	15
18,000 and Over	70	1	90	2
Total	4,670	100	4,840	100
	Truck Tandem Axles			
Under 16,000	200	19	470	26
16,000 - 19,999	90	9	170	9
20,000 - 23,999	140	14	190	11
24,000 - 27,999	200	19	300	17
28,000 - 31,999	350	34	600	33
32,000 and Over	50	5	70	4
Total	1,030	100	1,800	100

Location	Total Daily Vehicular Traffic	
	1953 Annual Average	1957 Summer Average
At Weight Station	6,100	8,300
At Test Project	9,750	16,200

1/ The numbers of axles at the test project in the heavier weight-groups are believed to be about the same as shown for the weight-station location. Northbound and southbound loadings are believed to be about equal.

heavy-truck axle loads are numerically approximately equal at the two locations.

With recognition of the foregoing explanation, the data of Table 2 are presented as an indication of the severity of axle loads to which the test sections have been subjected. Figures presented are those derived for the weight-station location, and they are without alteration to reflect differences in traffic volumes at the two locations. Appropriate 24-hr total traffic volumes over the test project, and at the weight station are given in the table to indicate differences in magnitudes of total-traffic flow.

CONSTRUCTION

The contract for the construction of Section 110X-5 was awarded in July 1951 and construction was started soon thereafter. Most of the grading, culvert construction, and part of the placing of the granular subbase was completed during the 1951 construction season. After a winter shutdown, work was resumed in the spring of 1952. Actual paving of the test sections was begun on May 21, and completed on July 25, 1952.

The new pavement was opened to traffic on July 22 and carried two-way traffic for about three weeks while the old pavement was being widened and resurfaced. Since August 9, 1952 the test sections have carried only the southbound traffic.

Grading

An embankment cross-section prevails through most of the project length because the finished grade of the pavement is generally from 1 to 3 ft above the relatively flat natural ground surface. Near the south end, a cut having a maximum depth of 5 to 6 ft extends about 1,000 ft in length, and terminates at a short, heavy fill (maximum height 14 ft) near Lake Springfield bridge. These are the only cuts and fills of consequence on the project.

The Illinois standard specifications, which governed the compaction of embankment for this project, required that construction methods be employed to secure not less than 90 percent of the maximum density shown on the wet-weight curve determined by the AASHO standard density test. The standard specifications further required that the moisture content of the material being placed should not exceed 110 percent of wet optimum.

Soil samples were secured from the compacted earth underneath the subbase and in-place density tests were made at 30 locations immediately ahead of paving operations. The samples, which were taken between Stations 91+00 and 264+65, represent the compacted earth to a depth of 6 in. The subbase material was sampled, and in-place subbase densities were determined, at the same locations. Test results of samples of the compacted earth are given in Table 3. These results substantiate those of the earlier soil tests made prior to construction. The soils are predominately dark colored silty clays of AASHO classification A-4(8) and A-6(8 to 12). Plasticity indexes range from 6 to 21 with a mean value of approximately 15. It is noteworthy that the uniformity of soil in this area is a circumstance favorable to the research that has been undertaken.

Field-density measurements of the compacted subgrade soil were made by the sand-hole method (AASHO Designation T-147-49) at the locations where the 30 samples were taken for which data are given in Table 3. Results in these field-density tests appear in Table 4 in combination with laboratory test data. Moisture-density tests were performed in the laboratory on nine of the 30 samples, these nine being chosen as representative of the range of the entire group of samples on the basis of grain size and Atterberg limits. Results of tests of the nine samples appear in Table 4. It will be seen from the table that relative densities are indicated to range from 92.5 to 105.8 percent, with a mean value of 98.2 percent on the wet-curve basis. The densities on the dry-curve basis range from 88.1 to 105.6 percent, with a mean value of 96.1 percent. The field water content of most of the samples was generally higher than optimum. The soil samples were taken and the in-place densities determined immediately prior to paving operations in the late spring of 1952, several months after the major portion of the embankment had been placed. The subbase was in place at the time of sampling and testing of the earth subgrade.

TABLE 3

PHYSICAL CHARACTERISTICS AND CLASSIFICATION OF SUBGRADE SOIL SAMPLES 1/

Sample Number	Station	Color	Soil Type	Group Classification	Mechanical Analysis Material Passing Sieves					Grain Size			Atterberg Limits		
					No. 10 (percent)	No. 20 (percent)	No. 40 (percent)	No. 100 (percent)	No. 200 (percent)	Sand 2.0 to 0.05mm (percent)	Silt 0.05 to 0.005mm (percent)	Clay < 0.005mm (percent)	LLL	LPL	PI
1	264+65	Brown	SiC	A-4(8)	-	100	99	96	90	10	60	30	24	18	6
2	263+50	Gray-Br.	SiC	A-6(9)	100	99	98	95	94	6	55	39	31	18	13
3	263+00	Gray-Br.	SiC	A-6(12)	100	99	98	95	95	6	56	38	36	17	19
4	262+00	Black	SiC	A-4(8)	100	99	98	96	95	5	60	35	31	21	10
5	261+00	Black	SiC	A-6(9)	100	99	98	96	95	5	60	35	32	19	13
6	260+00	Black	SiC	A-6(10)	-	100	99	97	97	5	56	39	34	18	16
7	259+00	Black	SiC	A-6(10)	100	99	97	94	94	6	57	37	35	19	16
8	258+00	Black	SiC	A-6(12)	100	99	98	96	96	9	52	39	38	18	20
9	264+90	Brown	SiCL	A-4(8)	100	98	97	94	93	10	61	29	27	19	8
10	257+00	Br.-Black	SiC	A-6(11)	-	100	99	97	97	5	59	36	35	17	18
11	256+00	Gr.-Brown	SiC	A-6(12)	100	99	98	96	95	7	57	36	38	18	20
12	255+00	Gr.-Brown	SiC	A-6(11)	100	98	96	93	93	10	53	37	37	19	18
13	254+00	Black	SiC	A-6(12)	100	99	99	98	97	6	58	36	39	20	19
14	250+00	Black	SiC	A-4(8)	-	100	99	96	96	9	61	30	27	17	10
15	244+00	Br.-Black	SiC	A-6(10)	100	99	98	96	95	6	55	39	37	21	16
16	234+50	Black	SiC	A-6(9)	-	100	98	97	97	7	56	37	34	22	12
17	220+00	Br.-Black	SiC	A-6(9)	100	99	99	97	97	11	59	30	34	22	12
18	182+00	Br.-Black	SiC	A-6(9)	100	98	97	95	94	8	52	40	35	23	12
19	168+00	Br.-Black	SiC	A-6(9)	100	99	98	96	96	7	53	40	36	23	13
20	164+00	Br.-Black	SiC	A-6(10)	-	100	99	98	97	5	57	38	36	22	14
21	160+00	Br.-Black	SiC	A-6(11)	-	100	99	98	98	4	57	39	38	20	18
22	155+00	Black	SiC	A-6(10)	-	100	99	98	98	7	54	39	37	21	16
23	150+00	Black	SiC	A-6(10)	100	99	96	93	92	15	50	35	34	20	14
24	145+50	Black	SiC	A-6(10)	100	99	98	96	96	9	52	39	37	22	15
25	132+50	Black	SiC	A-6(12)	100	97	93	88	87	13	51	36	39	18	21
26	118+25	Black	Clay	A-6(10)	100	99	96	91	91	15	48	37	39	23	16
27	113+00	Black	SiC	A-6(9)	100	99	97	94	93	11	52	37	35	23	12
28	106+00	Black	SiC	A-6(10)	100	99	99	97	96	6	54	40	38	23	15
29	98+00	Black	SiC	A-6(8)	-	100	99	97	96	6	57	37	32	21	11
30	91+00	Black	SiC	A-6(11)	100	99	98	95	94	7	56	37	40	22	18

1/ Samples include only materials in top of subgrade at average depth range of 0 to 6 inches.

TABLE 4

MOISTURE-DENSITY RELATIONSHIPS OF SUBGRADE SOIL SAMPLES 1/

Sample Number	Station	Soil Type	Field Water Content (percent)	Wet Curve Basis: Field Density (p.c.f.)	Wet Curve Basis: Optimum Moisture (percent)	Wet Curve Basis: Maximum Density (p.c.f.)	Wet Curve Basis: Relative Density (percent)	Wet Curve Basis: Relative Moisture Content (percent)	Dry Curve Basis: Field Density (p.c.f.)	Dry Curve Basis: Optimum Moisture (percent)	Dry Curve Basis: Maximum Density (p.c.f.)	Dry Curve Basis: Relative Density (percent)	Dry Curve Basis: Relative Moisture Content (percent)
1 2/	264+65	SiC	20.6	120.5	15.8	130.3	92.5	130.4	99.9	14.4	113.4	88.1	143.3
2 2/	263+50	SiC	23.3	125.5	19.2	124.3	101.0	121.4	101.8	17.3	105.7	96.3	134.7
3	263+00	SiC	24.0	126.3	(20.2)	(123.0)	102.7	118.8	101.9	(18.8)	(103.0)	98.9	127.7
4 2/	262+00	SiC	13.6	125.5	19.0	123.7	101.5	71.6	110.5	18.1	104.6	105.6	75.1
5 2/	261+00	SiC	23.1	126.0	20.2	124.2	101.4	114.4	102.4	17.8	104.8	97.7	129.8
6	260+00	SiC	23.7	126.0	(20.2)	(123.0)	102.4	117.3	101.9	(18.8)	(103.0)	98.9	126.1
7	259+00	SiC	24.9	122.4	(20.2)	(124.2)	98.6	123.3	98.0	(17.8)	(104.8)	93.5	139.9
8 2/	258+00	SiC	21.9	114.6	20.5	123.2	92.6	106.8	94.0	19.4	102.8	91.4	112.9
9	264+90	SiCL	20.2	121.2	(15.8)	(130.3)	93.0	127.8	100.8	(14.4)	(113.4)	88.9	140.3
10	257+00	SiC	17.7	121.5	(20.2)	(123.0)	98.8	87.6	103.2	(18.8)	(103.0)	100.2	94.1
11	256+00	SiC	21.2	116.0	(20.5)	(123.2)	93.8	103.4	95.7	(19.4)	(102.8)	93.1	109.3
12	255+00	SiC	16.5	119.7	(20.2)	(123.0)	97.3	81.7	102.7	(18.8)	(103.0)	99.7	87.8
13	254+00	SiC	17.3	120.0	(20.2)	(123.0)	97.6	85.6	102.3	(18.8)	(103.0)	99.3	92.0
14	250+00	SiC	20.0	126.2	(19.0)	(123.7)	102.0	105.3	105.2	(18.1)	(104.6)	100.6	110.5
15	244+00	SiC	28.0	116.5	(20.2)	(123.0)	94.7	138.6	91.0	(18.8)	(103.0)	88.3	148.9
16	234+50	SiC	14.6	122.5	(20.9)	(123.1)	99.5	69.9	106.9	(20.1)	(102.3)	104.5	72.6
17	220+00	SiC	26.8	117.7	(20.9)	(123.1)	95.6	128.2	92.8	(20.1)	(102.3)	90.7	133.3
18 2/	182+00	SiC	27.9	116.9	20.9	123.1	95.0	133.5	91.4	20.1	102.3	89.3	138.8
19	168+00	SiC	27.0	115.3	(20.9)	(123.1)	93.7	129.2	90.8	(20.1)	(102.3)	88.8	134.3
20	164+00	SiC	22.6	121.5	(23.7)	(119.8)	101.4	95.4	99.1	(21.9)	(97.8)	101.3	103.2
21 2/	160+00	SiC	23.4	120.9	20.2	123.0	98.3	115.8	98.0	18.8	103.0	95.1	124.5
22	155+00	SiC	27.0	117.9	(23.7)	(119.8)	98.4	113.9	92.8	(21.9)	(97.8)	94.9	123.3
23	150+00	SiC	26.4	118.3	(23.7)	(119.8)	98.7	111.4	93.6	(21.9)	(97.8)	95.7	120.5
24 2/	145+50	SiC	26.7	115.7	23.7	119.8	96.6	112.7	91.3	21.9	97.8	93.4	121.9
25	132+50	SiC	25.4	126.6	(20.5)	(123.2)	102.8	123.9	101.0	(19.4)	(102.8)	98.2	130.9
26	118+25	Clay	24.2	115.2	(23.7)	(119.8)	96.2	102.1	92.8	(21.9)	(97.8)	94.9	110.5
27	113+00	SiC	18.6	118.0	(20.9)	(123.1)	95.9	89.0	99.5	(20.1)	(102.3)	97.3	92.5
28	106+00	SiC	24.7	126.8	(23.7)	(119.8)	105.8	104.2	101.7	(21.9)	(97.8)	104.0	112.8
29 2/	98+00	SiC	20.3	123.0	20.6	122.0	100.8	98.5	102.2	19.9	102.0	100.2	102.0
30	91+00	SiC	25.6	120.3	(20.2)	(123.0)	97.8	126.7	95.8	(18.8)	(103.0)	93.0	136.2

1/ Samples taken at average depths of 0 to 6 in. below subbase.

2/ Moisture-density tests were performed in laboratory on these 9 samples chosen as representative of the entire group of samples on the basis of grain-size analysis and Atterberg limits. Values enclosed in parentheses are from laboratory test of one of the nine samples most nearly representative of the respective sample.

Subbase

A trench-type, undrained subbase, extending 1 ft beyond the pavement at each edge and having a design compacted thickness of 6 in., was used throughout the experimental project. The Illinois specifications for granular subbase at the time this project was constructed required that the material be uniform, and conform to the following gradation:

Percent Passing Sieves (Square Openings)			
3 in.	No. 4	No. 50	No. 200
100	45-90	5-25	5-10

There was no specification regarding the plasticity of the material.

The information regarding tests of granular subbase (Tables 5 and 6) related to 30 samples of such material, each sample taken and each density test made at the same location as the correspondingly numbered sample of subgrade soil given in Table 4.

Subbase material placed in the fall of 1951 extended from Station 113+00 to Station 121+00 and from Station 167+00 to Station 265+01. Two materials of somewhat different gradation were used in these areas. A bank-run gravel was first placed in a lift of varying thickness but averaging about 4 in. On top of this was placed a lift of coarse sand to complete the specified 6-in. total thickness. Subbase for the remainder of the experimental area was placed in 1952. This consisted of a single 6-in. course of bank-run gravel, spread and compacted immediately ahead of the paving operation.

At the time the samples reported in Tables 5 and 6 were obtained, the stratification of the two layers of slightly different material placed in 1951 could be detected in only a few instances. Presumably, considerable mixing of the two materials took place during the construction operation.

All samples of subbase material taken in areas of 1951 construction were nonplastic. Most, but not all, of the samples taken in areas of 1952 construction showed a small amount of plasticity. Physical characteristics determined for the samples are given in Table 5. All samples classified as either A-1-a(0) or A-1-b(0) sandy gravels.

The results of the field-density and field-moisture determinations, and the moisture-density relationships for the subbase samples are given in Table 6. Actual laboratory density tests were made on composite samples consisting of three or more of the field samples. Six groupings of 29 samples were made based on similarities of gradation and Atterberg-limit values. The relative densities averaged 91.5 percent on the wet-curve basis and 92.2 percent on the dry-curve basis. Relative densities were not determined for locations where two layers of material were clearly visible because the field work did not include a separate determination of the density of each layer.

The material having some plasticity appeared to compact more easily and to produce a more stable subbase with respect to supporting construction equipment than did the nonplastic material. This latter material remained loose on top even after several passes of the rubber-tired rollers.

Paving Procedures and Curing

Batch trucks traveling on the shoulder transported materials to the paver which was also operated on the shoulder. Paving progressed full width from south to north. The paving train for the period from May 21, 1952 through June 3, 1952 consisted of one paver followed by a mechanical concrete spreader, a finishing machine, and a mechanical longitudinal float. On June 4, 1952 a second paver and mechanical spreader were added to the paving train. After the passage of the mechanized equipment, hand straightedging, belting and brooming completed the surface-finishing operation except for a length of 3,628 ft of pavement at the north end where a burlap-drag finish was substituted for the broom finish. The burlap-drag finish was produced by two passes of wetted burlap over the surface of the concrete following belting. The change to the burlap drag at the north end of the project came about as the result of a general change in construction procedure throughout the state.

TABLE 5

PHYSICAL CHARACTERISTICS AND CLASSIFICATION OF GRANULAR SUBBASE SAMPLES 1/

Sample Number	Station	Subbase Placed 2/	Material Type	Group Classification	Mechanical Analysis, Passing Sieves: 3"	No. 4	No. 50	No. 200	Atterberg Limits: LLL	LPL	PI
				Specifications	100 (percent)	45-90 (percent)	5-25 (percent)	5-10 (percent)			
1	264+65	F 51	Sandy Gravel	A-1-a(0)	100	68	18	6			NP
2	263+50	F 51	"	A-1-b(0)	100	72	14	6			NP
3	263+00	F 51	"	A-1-b(0)	100	77	18	5			NP
4	262+00	F 51	"	A-1-b(0)	100	89	18	5			NP
5	261+00	F 51	"	A-1-b(0)	100	86	17	4			NP
6	260+00	F 51	"	A-1-b(0)	100	77	17	5			NP
7	259+00	F 51	"	A-1-b(0)	100	69	16	6			NP
8	258+00	F 51	"	A-1-a(0)	100	73	19	6			NP
9	264+90	F 51	"	A-1-b(0)	100	85	16	4			NP
10	257+00	F 51	"	A-1-b(0)	100	77	19	6			NP
11	256+00	F 51	"	A-1-b(0)	100	76	20	7			NP
12	255+00	F 51	"	A-1-b(0)	100	76	21	10			NP
13	254+00	F 51	"	A-1-b(0)	100	78	19	7			NP
14	250+00	F 51	"	A-1-b(0)	100	91	25	13			NP
15	244+00	F 51	"	A-1-b(0)	100	88	28	15			NP
16	234+50	F 51	"	A-1-a(0)	100	72	12	5			NP
17	220+00	F 51	"	A-1-b(0)	100	71	18	8			NP
18	182+00	F 51	"	A-1-b(0)	100	73	19	11			NP
19	168+00	F 51	"	A-1-a(0)	100	66	15	8			NP
20	164+00	S 52	"	A-1-b(0)	100	87	25	16	16	11	5
21	160+00	S 52	"	A-1-b(0)	100	85	21	13	16	11	5
22	155+00	S 52	"	A-1-b(0)	100	85	22	13	16	12	4
23	150+00	S 52	"	A-1-b(0)	100	85	26	17	16	14	2
24	145+50	S 52	"	A-1-b(0)	100	89	19	12			NP
25	132+50	S 52	"	A-1-a(0)	100	62	13	6			NP
26	118+25	F 51	"	A-1-b(0)	100	87	19	12			NP
27	113+00	F 51	"	A-1-b(0)	100	92	26	13			NP
28	106+00	S 52	"	A-1-b(0)	100	85	18	10			NP
29	98+00	S 52	"	A-1-b(0)	100	86	24	15	16	11	5
30	91+00	S 52	"	A-1-b(0)	100	88	26	15	17	13	4

1/ Samples include subbase material from an average depth of 0 to 6 in. below the bottom of the pavement

2/ F 51 - indicates fall of 1951. S 52 - indicates summer of 1952.

TABLE 6

MOISTURE-DENSITY RELATIONSHIPS OF GRANULAR SUBBASE SAMPLES 1/

Sample Number	Station	Material Type	Field Water Content	Wet Curve Basis: Field Density	Wet Curve Basis: Optimum Moisture	Wet Curve Basis: Maximum Density	Wet Curve Basis: Relative Density	Wet Curve Basis: Relative Moisture Content	Dry Curve Basis: Field Density	Dry Curve Basis: Optimum Moisture	Dry Curve Basis: Maximum Density	Dry Curve Basis: Relative Density	Dry Curve Basis: Relative Moisture Content
			(percent)	(p.c.f.)	(percent)	(p.c.f.)	(percent)	(percent)	(p.c.f.)	(percent)	(p.c.f.)	(percent)	(percent)
1	264+65	Sandy Gravel	7.0	89.9					84.0				
2	263+50	"	7.4	90.4	9.1	138.4	65.3	81.3	84.2	8.9	127.1	66.2	83.1
3	263+00	"	5.5	126.3	9.1	138.4	91.3	60.4	119.7	8.9	127.1	94.2	61.8
4	262+00	"	9.7	124.7					113.7				
5	261+00	"	13.9	124.1					109.0				
6	260+00	"	6.0	128.0	9.1	138.4	92.5	65.9	120.8	8.9	127.1	95.0	67.4
7	259+00	"	9.7	129.4					118.0				
8	258+00	"	7.1	126.3	9.6	141.6	89.2	74.0	117.9	9.1	129.6	91.0	78.0
9	264+90	"	17.2	125.1					106.7				
10	257+00	"	9.6	134.6	9.1	139.4	96.6	105.5	122.8	8.8	127.7	96.2	109.1
11	256+00	"	12.3	140.0	9.1	139.4	100.4	135.2	124.7	8.8	127.7	97.7	139.8
12	255+00	"	6.0	143.6	9.1	139.4	103.0	65.9	135.5	8.8	127.7	106.1	68.2
13	254+00	"	7.9	85.7					79.4				
14	250+00	"	10.6	140.5	9.8	137.1	102.5	108.2	127.0	9.5	125.0	101.6	111.6
15	244+00	"	7.3	130.6	9.8	137.1	95.3	74.5	121.7	9.5	125.0	97.4	76.8
16	234+50	"	10.3	133.3	9.6	141.6	94.1	107.3	120.9	9.1	129.6	93.3	113.2
17	220+00	"	7.9	140.6	9.8	137.1	102.6	80.6	130.3	9.5	125.0	104.2	83.2
18	182+00	"	4.2	128.6	9.0	137.2	93.7	46.7	123.4	8.7	126.2	97.8	48.3
19	168+00	"	5.7	131.1	9.6	141.6	92.6	59.4	124.0	9.1	129.6	95.7	62.6
20	164+00	"	7.5	127.2	9.0	137.2	92.7	83.3	118.3	8.7	126.2	93.7	86.2
21	160+00	"	5.3	117.0	9.0	137.2	85.3	58.9	111.1	8.7	126.2	88.0	60.9
22	155+00	"	7.3	128.6	9.0	137.2	93.7	81.1	119.9	8.7	126.2	95.0	83.9
23	150+00	"	4.7	122.6	9.0	137.2	89.3	52.2	117.1	8.7	126.2	92.8	54.0
24	145+50	"	7.3	108.9					101.5				
25	132+50	"	7.5	128.4	9.6	141.6	90.7	78.1	119.4	9.1	129.6	92.1	82.4
26	118+25	"	11.0	131.5	9.8	137.1	95.9	112.2	118.5	9.5	125.0	94.8	115.8
27	113+00	"	9.2	126.4	9.8	137.1	92.2	93.9	115.8	9.5	125.0	92.6	96.8
28	106+00	"	8.6	124.8	9.8	137.1	91.0	87.8	114.9	9.5	125.0	91.9	90.5
29	98+00	"	8.6	130.3	9.0	137.2	95.0	95.6	120.0	8.7	126.2	95.1	98.9
30	91+00	"	11.1	136.9	9.0	137.2	99.8	123.3	123.2	8.7	126.2	97.6	127.6

1/ Laboratory moisture-density tests were performed on combinations of three or more samples grouped on the basis of similarities of gradation and Atterberg limits. Application of the laboratory test results has been omitted in cases of samples 1, 4, 5, 7, 9, 13 and 24 because, at the time of field sampling, the subbase material showed two distinct cources which were not tested separately.

Where mesh reinforcement was used, the concrete was placed in two lifts. When one mixer and spreader were in use, the first lift was placed and struck off about 2½ in. below the top of the forms for a distance of 50 to 60 ft ahead of the finishing machine. The mixer and spreader were then backed while the mesh was being installed, and the top lift of concrete was placed and spread. With two mixers and spreaders in use, the bottom lift was placed by the lead mixer and struck off by the lead spreader, and the top lift was placed with the second mixer and struck off with the second spreader. This latter procedure was followed in general for certain test sections where mesh was omitted, but the lifts placed were more nearly equal in thickness, permitting the second mixer to operate nearer its full capacity.

The concrete was consolidated from the surface by a pan-type vibrator mounted at the rear of the mechanical spreader.

In paving adjacent to the full-depth metal-plate contraction joints, the concrete was placed carefully on each side of the metal plate and hand-spaded into place to hold the plate in correct alignment. The spreader was raised over the plate and the pins holding the plate cap were removed after the passage of the spreader over the joint. The front screed of the finishing machine was also raised over the plate to prevent displacement. At locations where the joints were to be edged, the plate cap was left on until after the passage of the longitudinal float. At locations where edging was to be omitted, the plate cap was removed ahead of the longitudinal float.

In paving over the basket assemblies holding the load-transfer dowels, at locations of dummy-groove and sawed joints, the concrete was placed directly over the basket assemblies and no special precautions were required for further spreading and finishing operations. The dummy-groove joints were formed by forcing a steel bar having a T-shaped cross-section into the fresh concrete. These joints were hand-finished with an edging tool after the concrete had attained proper stiffness. The locations of the center of the dowels were marked on the forms and later in the fresh concrete at each pavement edge to accurately position the dummy-groove and sawed contraction joints.

Vibrators operating from the lead spreader were used to consolidate the concrete around assemblies of dowel bars at transverse joints and tie bars of the longitudinal center joint, and along the forms.

The impermeable-paper method of curing was used on all but one of the test sections. The pavement of the section on which impermeable paper was not used was cured with wetted burlap. Full-width paper was placed on the pavement after the surface moisture had disappeared and the concrete had stiffened sufficiently to support the paper without being marred. Extra strips, 2 ft wide, were placed along each edge and inserted under the full-width paper. These were pulled down over the pavement edges after the forms were removed. The paper remained in place for a minimum period of 72 hours after placement of the concrete. On the section where wetted burlap was substituted for the paper (Section 6), two thicknesses of burlap were applied and saturated as soon as the concrete had stiffened sufficiently to permit such application without marring the surface. The burlap was kept thoroughly saturated by periodic sprinkling, and remained in place for a minimum period of 72 hours.

On Sections 3 and 6 having sawed joints spaced at 100-ft intervals, the initial saw cut was made on the day following paving (two joints were sawed four days following paving) by one pass of a single-bladed machine. A diamond-type saw blade was used in the cutting operation and cut a kerf of about ⅛ in. width. Initial cuts were made to depths of 2¼ and 3 in. The saw groove was widened at a later date by an additional parallel cut to a 1½ in. depth. The final grooves were about ½ in. in width.

Although the sawing machine that was used was somewhat primitive when compared with those used in present-day construction, the operation was the same as that in general use today and no particular problems were involved.

Mix Design, Paving Control, Tests and Test Specimens

The coarse aggregates for the pavement consisted of crushed limestone, having a specific gravity of 2.65. The typical gradation for each size was as follows:

	Percent Passing Sieves (Square Openings)					
	2½ in.	2 in.	1½ in.	1 in.	½ in.	No. 4
Size A	100	97	52	6	2	
Size B				100	40	3

The fine aggregate consisted of natural sand, having a specific gravity of 2.65. The typical gradation was as follows:

Percent Passing Sieves (Square Openings)				
⅜ in.	No. 4	No. 8	No. 16	No. 50
100	97	83	72	18

Type 1A cement was used throughout.

The concrete mix was designed using the mortar-voids theory as normally applied by Illinois. The cement factor ranged from 1.42 to 1.45 bbl of cement per cu yd of concrete, with 1.44 being used for the major portion of the test pavement. Approximately 5.4 gallons of water per sack of cement provided satisfactory workability. The mix that may be considered as typical was as follows:

Cement	-	94 pounds
Sand	-	200 pounds
C.A. Size A	-	172 pounds
C.A. Size B	-	172 pounds
Water	-	5.4 gallons

Usual standard Illinois paving controls were used throughout the project.

Tests usually were made twice daily to determine the percentage of entrained air in the concrete. Sampling and testing were essentially the same as those of the Test for Air Content of Freshly Mixed Concrete by the Pressure Method—AASHO Designation T 152-53. The consistency of the freshly mixed concrete was measured by the standard slump test. The results of the air-content tests and the slump tests are reported in Table 7. The air content of the plastic concrete averaged 3.9 percent and slump averaged 3.1 in.

Fifty 6- by 6- by 30-in. beam specimens were made during the placing of the test pavement. Four beams were made per day during the first three days of paving. After the third day, two per day were made. Beams were cast from freshly mixed concrete representative of the mixture and covered with wetted burlap. As soon as the beam forms could be removed and the beams could be transported without damage, they were taken to a sand pit and cured in damp sand until tested for flexural strength. Testing was performed on a modified cantilever-type machine, employing the center-point method of loading. When four beams were being made per day, they were tested usually at ages of 3, 5, 7 and 14 days. When only two beams were made per day, testing was normally at the ages of 7 and 14 days. The results of 7- and 14-day beam tests are given in Table 7. The average modulus of rupture at age 14 days was 749 psi.

A total of 44 cores were drilled from the hardened concrete before the pavement was opened to traffic. These specimens were 4½ in. in diameter and are drilled primarily as a check on pavement thickness. The core lengths indicate that all pavement equaled or slightly exceeded the specified thicknesses. The cores were tested for compressive strength on July 17, 1952 at an average age of approximately 40 days. The locations at which the cores were drilled and the results of the tests are given in Table 8. The average value of compressive strength was 4,433 psi, and the maximum and minimum values were 6,434 and 3,489 psi, respectively.

RESEARCH PROCEDURE

It has been mentioned previously that the controls exercised over the construction operations differed in no way from usual Illinois practice on any normal construction project. Supervision and routine inspection were handled by the resident engineer and his assistants. Special observations to obtain information likely to be of aid in inter-

TABLE 7

PORTLAND CEMENT CONCRETE MIX CONTROL TEST DATA

Stations of Test Samples & Beam Specimens	Entrained Air	Slump	Modulus of Rupture	
			7 Days	14 Days
	(percent)	(inches)	(psi)	(psi)
262+15	4.9	5.0	613	585
253+00	4.7	2.0	549	841
246+00	4.7	3.3	643 1/	775
237+50	4.7	2.5	664 2/	809
227+50	4.3	2.9	689	858
217+00	3.0	4.0	626	700
208+25	3.7	2.8	709	794
198+00	3.7	2.5	649	740
183+80	4.3	3.5	598	755
170+50	3.2	3.0	625	694
160+20	3.1	3.5	600	721
147+40	3.5	3.5	631	781
132+80	3.9	3.0	690	751
131+16	3.2	3.0	561	723
126+10	3.2	2.8	615	714
112+50	3.9	3.0	760	733
96+00	4.5	3.0	648	773
80+25	3.8	3.2	585	719
63+75	4.2	3.5	649	735
58+40	4.4	3.0	723	812
51+75	3.9	3.5	580	685
44+48	4.2	3.0	655 3/	790
Average	3.9	3.1	639	749

1/ Tested at age 10 days. 2/ Tested at age 9 days 3/ Tested at age 5 days.

preting the results of research were made by a research engineer and in some instances by a materials engineer.

The only measurements entirely of a research nature that were made during construction were at special plugs installed in the concrete to furnish information on slab movements at joints. These are described in detail later.

Pavement Condition Surveys

Several periodic surveys of pavement behavior have been made since the construction of the test project. The first of these, which did not include Sections XA and XB, was completed immediately prior to the opening of the pavement to traffic on July 22, 1952. Follow-up surveys have been made once or twice a year since that date and have included Sections XA and XB.

The condition surveys have been made on foot, and the presence and location of such defects as have been found have been mapped. The only defects noted thus far have been cracks, spalls, faults, and shoulder holes. Because of the significance of spall in assessing the relative behavior of the experimental joint installations, the amount and degree of spall have been mapped in considerable detail.

The condition of the joint seal has also been observed and recorded during the condition surveys.

Measurement of Faults

A fault-measuring device (Fig. 4) which has been in use for several years in Illinois and which has been found to be superior to similar devices that have been employed

TABLE 8

COMPRESSIVE STRENGTH OF PAVEMENT CORES
(Approximate Average Age - 40 Days)

Station	Compressive Strength (psi)	Station	Compressive Strength (psi)
263+20	4587	141+35	3942
257+70	4354	138+40	4606
251+00	4134	132+60	4440
245+65	4585	126+40	6434
235+35	4856	121+00	4854
230+35	4662	115+90	5237
225+95	4263	109+75	4806
221+25	4186	105+64	4102
216+00	3489	103+65	4437
212+65	4122	102+00	3896
205+70	5163	98+15	5256
199+75	4874	94+30	4250
192+90	3721	90+65	3537
188+75	5123	86+90	4413
183+30	4154	84+15	4942
177+40	4288	79+85	4199
171+25	4005	74+95	4256
165+95	5170	70+35	4431
159+70	4334	65+45	5070
154+90	4134	64+15	3929
149+80	3918	60+85	4192
144+75	3918	44+00	3588
		Average, 44 cores	4433

previously was used to measure differential vertical slab displacement at the joints and cracks. This meter was developed after it was found that the devices previously used and supported by two legs on one side of the joint produced erroneous readings where the pavement surface at the joints was higher than the surrounding pavement—a condition which was found to be of frequent occurrence.

The two free-moving members, shown in Figure 4, are principal features that distinguish the new fault meter from earlier models with a single upright moving member. Whereas use of the earlier model required its placement on one slab with measurement made across the separation to the adjoining slab, the new device is placed astride of a joint or crack so that it rests on the two adjoining pavement slabs with supporting shoes approximately equidistant from the joint or crack being measured for faulting.

Fault meter surveys of the test sections have been made periodically since construction. Faults are measured in the wheel paths at all transverse joints and cracks.

Measurement of Slab Movements

Because a major phase of study under this research project relates to the design and performance of pavement slabs of different lengths, these being articulated by contraction joints of varying functional design in respect to slab-separation and load-transfer, it was decided to provide for making rather precise horizontal and vertical measurements at joints, and also at transverse cracks. Therefore, brass reference plugs were set at most of the joints, and soon after construction at most of the cracks that occurred. Cracking immediately following construction was confined to the unjointed sections.

The plugs are 3/8 in. in diameter by 1 in. long, each having a 1/8-in. diameter hole drilled at the center axis. These were installed in pairs at the joints and cracks where measurements were to be made. They were placed about 10 to 13 in. each side of joints and cracks and approximately 6 in. centerward from the (west) edge of the pavement.

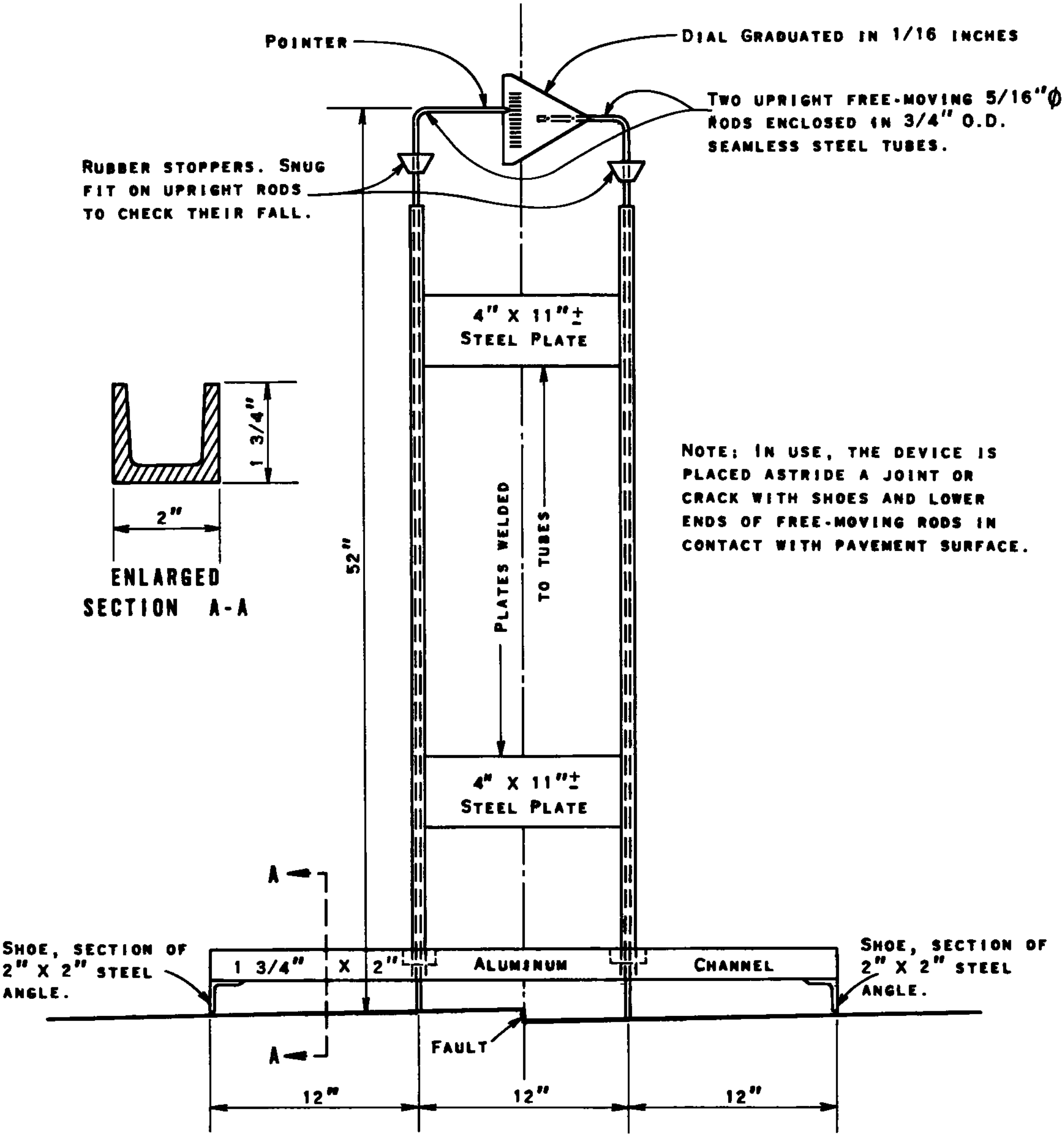


Figure 4. Mechanical principles of fault-measuring device.

Vertical placement is about ⅛ in. below the pavement surface for protection from snow-removal equipment. The reference plugs were installed in most instances in the plastic concrete. At the cracks, and at a few of the joints, they were installed in the hardened concrete using a star drill. Center holes of the plugs were closed against infiltering foreign material by copper rivets placed in the holes. Installations have been made at 172 transverse joints, and at 27 panel cracks.

Repetitive measurements have been made horizontally and vertically between the various sets of plugs using a specially-designed extensometer. By means of verniers, readings are made to 0.001 in. horizontally and to 0.01 in. vertically. Caliper points of the extensometer are cone-shaped, and are self-centering when placed in the ⅛-in. holes in the reference plugs. A level bubble is mounted on the extensometer, and the instrument is leveled at the time of reading. Recorded readings consist of the horizontal distance between plugs and the elevation differential of the tops of the plugs. Initial readings were made at each set of plugs as soon after installation as the concrete became firm. Horizontal readings indicate the opening and closing that takes place at the contraction joints and cracks, and the vertical readings are for correlation with faulting that may take place at the joints or cracks.

Following the initial recording of extensometer readings at the joints of each test section, an early series of readings was made at frequent but irregular intervals. Such early readings were continued until there was conclusive evidence of slab separation at each joint. Subsequent extensometer readings have been made at irregular intervals varying from 2½ to 17 months. Some of the longer intervals prevent much significance being derived from trends that can be established to date, but this may not be seriously disadvantageous in respect to long-term movements if the readings are continued at regular summer and winter intervals in the future.

Surface Smoothness

A road-smoothness indicator patterned after the Bureau of Public Roads' indicator (see "Standardizable Equipment for Evaluating Road Surface Roughness," Public Roads, Vol. 21, No. 12, February 1941) and constructed by the Illinois Division of Highways became available for use on the test pavement in July 1957.

Miscellaneous Research

As aids in understanding the test features a few comments regarding research procedure are as follows: (1) About one-half of the joints of Section 7 were sealed with a hot-application rubber-asphalt material, whereas the remaining joints of Section 7 and of all other test sections were sealed with a cold-applied rubber-asphalt material. It is noteworthy that, although the contract special provisions called for use of a double-boiler type of kettle for heating the hot-application compound, the manufacturer of the material, in personal charge of the use of his product on the test project, elected to forego the expense of providing the special equipment because of the small quantity of material to be used, finally amounting to about 1 cu ft. Application was therefore by a somewhat crude method, and it is not known whether test results were altered thereby. (2) A joint-grooving machine was usedto abrade and remove surface mortar from the opposing vertical faces of some of the hand-edged contraction joints prior to sealing. The abrading force was transmitted by means of a high-speed revolving drum guided above the road surface along the joints. Star-shaped, nonpowered cutting wheels, positioned around the drum-periphery, served as the abrading medium. Staggering of cutters permitted simultaneous abrading of both faces of a joint. Spacer-washers provided adjustment of abrading width. Effectiveness of the machine was impaired by some spalling of joints and by not achieving complete continuity of abrasion. There was a resulting belief that the particular unit of equipment was not entirely suitable for the purpose for which it was used.

EVALUATION OF OBSERVATIONS

The test pavements have served satisfactorily during seven years of use under normal traffic. There has been no pumping, likewise no scaling. Few shoulder holes have been in evidence. Cracks are infrequent, and those occurring are principally of the transverse type, although a few infiltration cracks are present here and there. (The term "infiltration" is used here to identify cracks that begin at transverse joints and transverse cracks most frequently from 1 to 3 ft in from the outside edge of the pavement, and extend in a generally longitudinal direction a few inches to sometimes several feet.) Faulting at transverse joints and cracks up to the present time is generally slight. Spalls are frequent at the joints and cracks of certain test sections, but are in but a few instances more than superficial in nature.

Observations of the experimentation, in cases of some items, began prior to the close of construction, and have continued periodically for all items of research. Such observations are discussed under topic headings that follow.

Cracks

Results of the most recent crack survey, made in 1959, are given in Table 9. As stated previously the only crack types that have appeared are transverse cracks and "infiltration" cracks.

It will be noted in Table 9 that the transverse cracks that have occurred during the first seven years of pavement life are few in number, except in the two test sections of plain concrete built without transverse joints. Even in these sections the average panel lengths formed by the cracks are after seven years within the general range of the panel lengths of the reinforced sections constructed with joints at 100-ft intervals. The fact that the average uncracked slab length for the 9-in. thick nonjointed pavement is slightly greater than that for the 10-in. thick nonjointed pavement is not considered to be significant at this time.

Details regarding the progression of transverse cracking that has taken place in the nonjointed and nonreinforced pavement test sections are presented in Table 10. It will be noted from the table that the bulk of the cracking that has taken place in the first seven years of pavement life occurred during the first eight months.

TABLE 9

CRACKS IN PAVEMENT, SHOULDER HOLES AND PUMPING

(1959 Survey)

Test Section	Distinctive Design Features	Section Length (feet)	Transverse Cracks (number)	Average Joint and Crack Interval (feet)	Infiltration Cracks (number)	Shoulder Holes 1/ (number)	Pumping (number)
X A	10-in. reinforced concrete; unedged metal-plate joints at 100 ft	3620	0	93	3	0	
X B	10-in. reinforced concrete; edged metal-plate joints at 100 ft	1561	1	82	0	0	
1A	10-in. plain concrete; no joints	1357	17	75	1	0	
1B	10-in. plain concrete; dummy-groove joints at 20 ft (no dowels)	1337	0	20	0	13	
2A	9-in. plain concrete; no joints	1353	15	85	3	5	None
2B	9-in. plain concrete; dummy-groove joints at 20 ft (no dowels)	1353	0	20	0	23	
3	10-in. reinforced concrete; sawed joints at 100 ft	991	2	71	1	3	
4	10-in. reinforced concrete; dummy-groove joints at 100 ft. (abraded faces)	3206	3	89	4	7	
5	10-in. reinforced concrete; dummy-groove joints at 100 ft	3203	9	73	1	6	
6	10-in. reinforced concrete; sawed joints at 100 ft	1001	2	83	0	0	
7	10-in. reinforced concrete; edged metal-plate joints at 100 ft (abraded faces)	3576	6	83	1	6	

1/ All shoulder holes adjacent to main travel lane

TABLE 10

TRANSVERSE CRACKS IN PLAIN CONCRETE NONJOINTED SECTIONS AT VARIOUS AGES

Age of Pavement	Section 1A, 10-in. Thick 1/		Section 2A, 9-in. Thick 2/	
	Passing Lane	Travel Lane	Passing Lane	Travel Lane
3 days	1	1	0	0
4 days	2	2	1	1
5 days	2	2	3	3
7 days	2	2	6	6
6 weeks	5	5	7	7
7 weeks	7	7	7	7
8 weeks	7	7	10	10
8 months	15	15	11	11
3 years	15	15	11	12
5 years	15	15	14	16
7 years	16	17	14	16

1/ Length 1357 feet; 1 construction joint
2/ Length 1353 feet; 1 construction joint

Infiltration cracks that have developed to date are relatively few in number (Table 9). Those that have appeared are barely visible and are not considered to be especially significant as evidence of deterioration. If the present trend toward a concentration of these cracks on the sections with transverse joints and cracks at the longer intervals continues, these will be considered to have significance in showing an association of this type of cracking with greater joint and crack openings and the probability of increased infiltration of foreign material in the openings. For the present, the evidence is not considered to be sufficient to draw any conclusions in this regard.

Spall at Joints and Cracks

Detailed measurements of spall at joints and cracks were made in 1956, four years after construction. In the field survey the distances that the spalls extended along the joints and the spalled widths were measured. In tabulating the data, the spalled areas were placed in four severity classes depending on the width of spall, as follows:

Class I - Up to and including ⅝ in. width.
Class II - Over ⅝ in. width up to and including 1¾ in. width.
Class III - Over 1¾ in. width up to and including 3 in. width.
Class IV - Over 3 in. width.

Data on the extent and severity of spall found in 1956 are presented in Table 11 where the information is tabulated on a per-1,000-lineal-feet-of-pavement basis for comparison. It will be seen from the table that the only significant spall is located at the unedged metal-plate joints and at the transverse cracks that have formed in this plain concrete pavements without joints.

Photographs showing typical conditions at joints and cracks in March 1954 at the end of two winters following construction are presented in Figure 5.

TABLE 11

SERVICE CONDITION OF TRANSVERSE JOINTS AND CRACKS

Test Section	Distinctive Design Features	(1956 Survey) Lineal Feet of Spall at Joints and Cracks Per 1000 Lineal Feet of Pavement: Class I	Class II	Class III	Class IV	Total	(1957 Survey) Percent of Original Seal Ruptured, Raveled or Missing: Ruptured in Place	Raveled or Missing	Total
X A	10-in. reinforced concrete; unedged metal-plate joints at 100 ft	11	40	30	23	104	0	100	100
X B	10-in. reinforced concrete; edged metal-plate joints at 100 ft	6	2	1	1	10	22	78	100
1A	10-in. plain conrete; no joints	24	17	4	7	52	-	-	-
1B	10-in. plain concrete; dummy-groove joints at 20 ft (no dowels)	Trace	1	3	1	5	86	14	100
2A	9-in. plain concrete; no joints	12	25	11	4	52	-	-	-
2B	9-in. plain concrete; dummy-groove joints at 20 ft (no dowels)	1	3	2	6	12	79	21	100
3	10-in. reinforced concrete; sawed joints at 100 ft	Trace	3	2	2	7	75	25	100
4	10-in. reinforced concrete; dummy-groove joints at 100 ft (abraded faces)	1	1	1	2	5	68	32	100
5	10-in. reinforced concrete; dummy-groove joints at 100 ft	1	1	1	0	3	60	40	100
6	10-in. reinforced concrete; sawed joints at 100 ft	0	Trace	0	0	Trace	87	13	100
7	10-in. reinforced concrete; edged metal-plate joints at 100 ft (abraded faces)	Trace	5	3	1	9	95	5	100
7 1/	10-in. reinforced concrete; edged metal-plate joints at 100 ft (abraded faces)	Trace	1	1	Trace	2	100	0	100

1/ Hot-poured rubber-asphalt seal

Note 1. The various classes of spall are defined in the text.

Note 2. A cold-applied rubber-asphalt sealing compound used on all joints except on part of Section 7 where a hot-poured rubber-asphalt sealing compound was used.

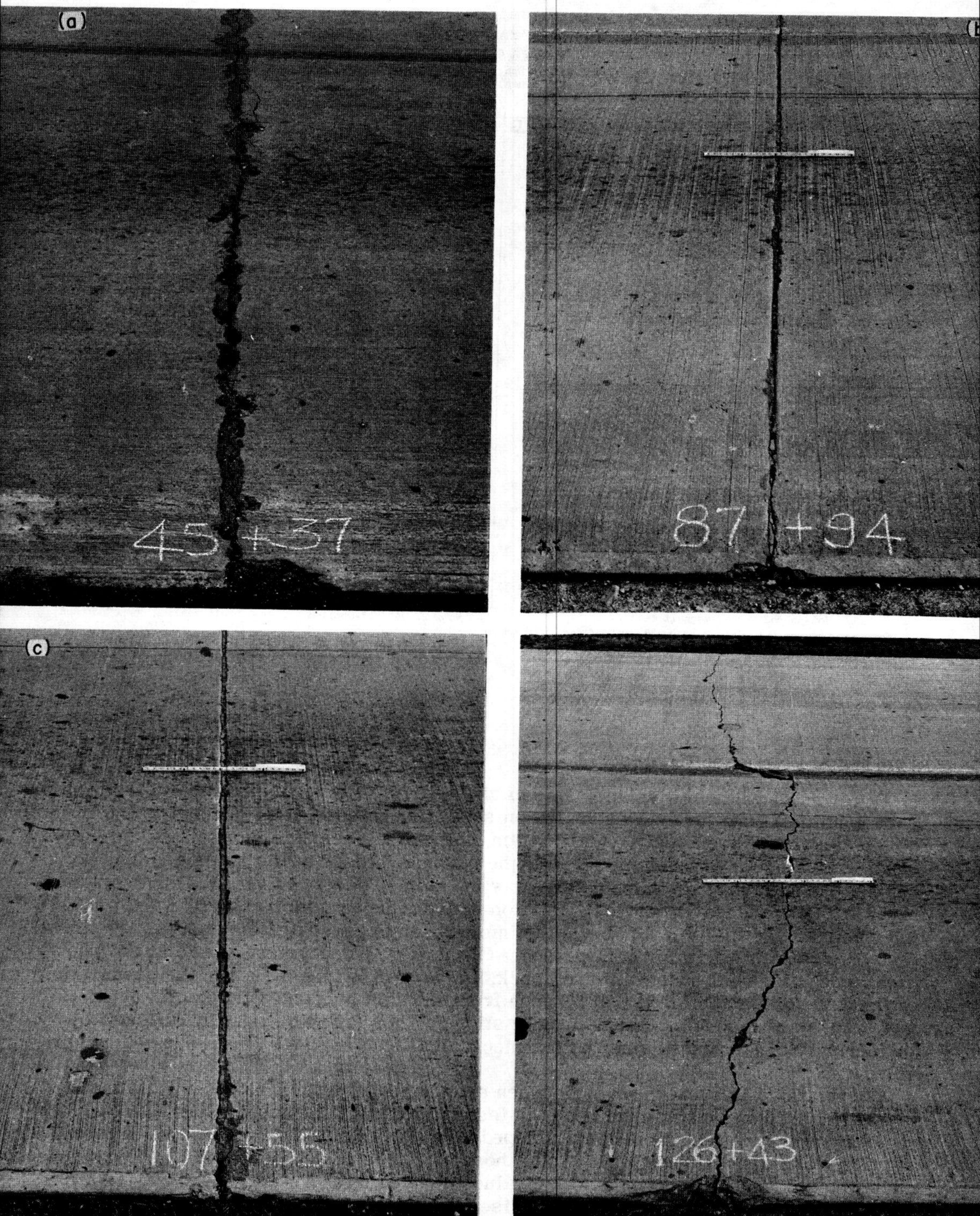
Faulting at Joints and Cracks

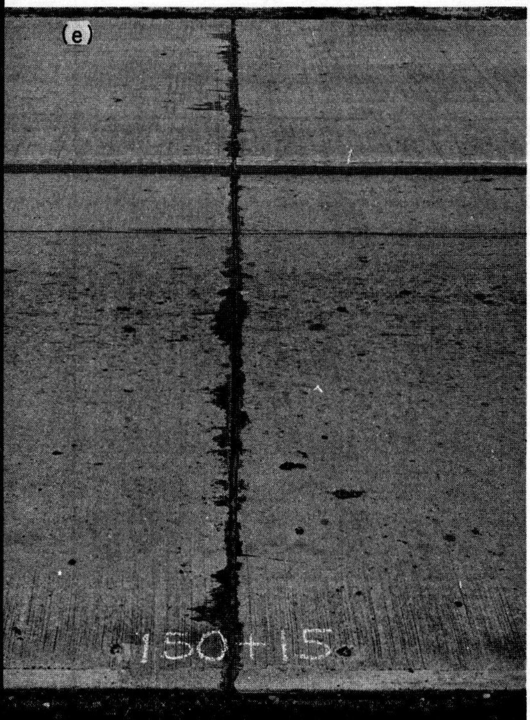
Differential vertical displacements of abutting slabs at joints and cracks (faulting) have been measured in the wheelpaths on several occasions in the manner and with the device described previously. Information on the displacements is also available from readings made on the brass plugs near the pavement edges at the joints and cracks.

The most recent survey of faulting was made in 1959, seven years after the pavement had been placed in service. Data on slab displacements that were noted in 1959 are summarized in Table 12 where the information is presented on a per-1,000-lineal-feet-of-pavement basis for comparison. Of the two measurements made in the wheelpaths at each lane at each joint and crack, the larger has been used in the preparation of the data for the table. It will be seen from the data presented that faults at joints and cracks are frequent, though usually slight. Thus far there is little difference in the severity of faulting whether or not mechanical load transfer devices have been provided.

Correlations have been made between extensometer determinations of the average displacement per joint or crack of each test section, and the average test-section faulting per joint or crack, the latter developed from faulting measurements at corresponding times and approximate locations to those of extensometer derivations. Maximum deviation per second between average values derived from the two sources was 0.02 in., with several sections showing precise agreement in values.

Although faulting at transverse joints and cracks was found nowhere to be significant, field survey parties have noted audible evidence of differential slab movements under traffic at the joints without mechanical load-transfer devices (Sections 1B and 2B). As heavier axle loads pass, there are distinct thumping and grinding sounds, indicating that independent movement rather than hinge-like action may be occurring between adjoining slabs.


(a)
45+37
87+94
(c)
107+55
126+43



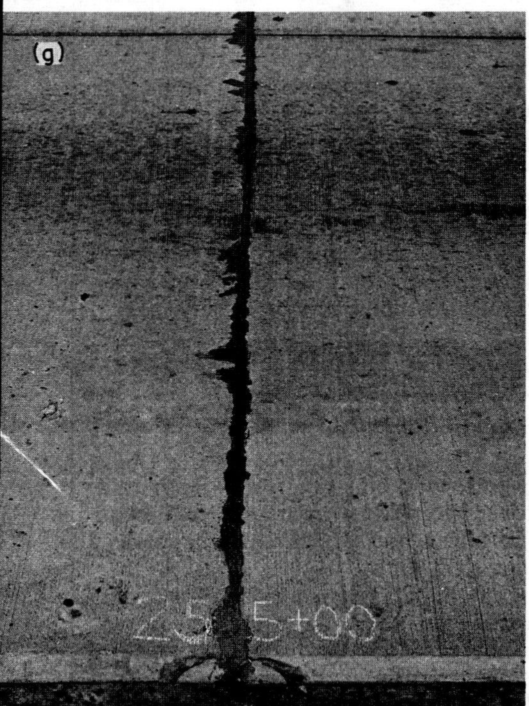


Figure 5. Typical joint and crack conditions two years after construction (March 1954), showing: (a) unedged metal-plate joint, 100-ft spacing, cold-applied sealing compound; (b) edged metal-plate joint, 100-ft spacing, cold-applied sealing compound; (c) dummy-groove joint, 20-ft spacing, cold-applied sealing compound; and (d) crack in nonjointed pavement, no seal applied. Figure 5 (Continued), showing: (e) sawed joint, ½-in. groove, 100-ft spacing, cold-applied spacing compound; (f) dummy-groove joint, 100-ft spacing, cold-applied sealing compound; and (g) dummy-groove joint, 100-ft spacing, hot-poured sealing compound.

TABLE 12

FAULTS AT JOINTS AND CRACKS AND ROAD ROUGHNESS INDEX

Test Section	Distinctive Design Features	(1959 Survey) Joints and Cracks Faulted Per 1000 Lineal Feet of Pavement: Travel Lane, 1/8 In. or More	Travel Lane, 1/4 In. or More	Travel Lane, 1/2 In. or More	Passing Lane, 1/8 In. or More	Passing Lane, 1/4 In. or More	Passing Lane, 1/2 In. or More	(1959 Survey) Road Roughness Index 1/: Travel Lane	Passing Lane
		(number)			(number)			(inches per mile)	
XA	10-in. reinforced concrete; unedged metal-plate joints at 100 ft	2	0	0	1	0	0	-	-
XB	10-in. reinforced concrete; edged metal-plate joints at 100 ft	3	0	0	1	0	0	86	83
1A	10-in. plain concrete; no joints	6	1	0	1	0	0	90	82
1B	10-in. plain concrete; dummy-groove joints at 20 ft (no dowels)	9	1	0	2	0	0	115	105
2A	9-in. plain concrete; no joints	7	1	0	2	0	0	85	82
2B	9-in. plain concrete; dummy-groove joints at 20 ft (no dowels)	28	2	0	2	1	0	108	91
3	10-in. reinforced concrete; sawed joints at 100 ft	1	0	0	1	0	0	82	84
4	10-in. reinforced concrete; dummy-groove joints at 100 ft (abraded faces)	1	0	0	1	0	0	79	85
5	10-in. reinforced concrete; dummy-groove joints at 100 ft	2	0	0	1	0	0	75	80
6	10-in. reinforced concrete; sawed joints at 100 ft	1	1	0	1	0	0	70	76
7	10-in. reinforced concrete; edged metal-plate joints at 100 ft (abraded faces)	6	0	0	3	0	0	77	89

1/ Determined with Bureau of Public Roads' type road-smoothness indicator

Surface Smoothness

The Illinois road-smoothness indicator did not become available for use on this project until 1957. The as-constructed road-roughness indexes for the test pavements are therefore not available. Smoothness determinations were made with the device in 1957, and again in 1959. Recorded values of the road-roughness index made in 1959 did not vary appreciably from those made in 1957. Those of 1959 are given in Table 12 where faulting data are also presented. Roughness indexes for the test sections constructed without joints and for those constructed with joints at 100-ft intervals range between 70 and 90 in. per mile. Roughness indexes for the pavements with dummy-groove joints at 20-ft intervals are somewhat higher and lie within the range of 91 to 115 in. per mile.

No consistent relationship between the measured depth at faults at joints and cracks and road-roughness index values is observable up to the present (Table 12).

Condition of Joint Seals

Rupture of the joint seals began the first winter following construction. Both cold-applied and hot-poured seals showed signs of rupture. Rupture was least in the sections of 20-ft joint spacing where only the cold-applied seal was used. This was probably the result of lesser movement at these more closely spaced joints.

The effects, if any, of abrading the joint faces are considered not to be distinguishable. As mentioned previously, the attempts to abrade the joint faces with the device at hand did not result in a uniform removal of the mortar film. Actually, no separation of a mortar film from the joint faces through adherence of the sealing material was found anywhere on this particular project, regardless of whether or not abrasion of the joint faces was attempted.

Summary data concerning the condition of the seals in the winter of 1957 following five years of service are presented in Table 11. It will be noted that the seal at all joints was found to be totally ruptured. Raveling of the seal was found to vary with the section but in most instances no distinct pattern is discernible. The hot-poured material, which was placed in joints with definite grooves or wells, was found to be raveled, whereas the cold-applied material placed under similar circumstances showed raveling in varying amounts. Cold-applied material placed on the unedged joints without grooves was totally raveled. Hot-poured material was not used in the unedged joints.

Reference is again made to Figure 5 where photographs of typical joint and crack conditions two years after construction are shown.

Information relative to measured horizontal movements of slab ends at joints and cracks is presented in the following section.

Joint and Crack Openings

As explained previously, brass reference plugs have been installed in pairs at joints and cracks for use in making precise measurements of vertical and horizontal slab movements. Most of the plugs at the joints were installed in the plastic concrete. All plugs at the cracks and a few at the joints were installed by drilling holes in the hardened concrete and seating the plugs in a mortar mixture. Initial extensometer readings of the distance between the plugs of each pair and of the difference in elevation between the plugs of each pair were made as soon as the plugs became firmly sealed. In the case of the plugs placed in the plastic concrete, readings were made at frequent, though irregular, intervals at each joint until it became apparent that the joint had opened. Readings were scheduled on a seasonal basis thereafter.

Analysis of changes in plug spacings has revealed two characteristics horizontal movements taking place at the joints. The first of the two movements, very slight but measurable, took place before the slab faces at the joints became visibly separated. Following these early movements, the long-term opening and closing cyclical movements became operative.

Early Slab Movements. –The frequent though irregular plug-spacing measurements that were made in the early hours and days following construction showed slight but measurable expansive and contractive movements to be taking place at the transverse joints prior to visible separation of the slabs. Plugs spaced about 24 in. apart frequently showed decreases from the original spacing in the order of 0.003 in., and increases in the order of 0.02 to 0.03 in. before visible separation occurred. Refinements adopted in planning and executing the research on this project were not adequate for following these movements with precision.

Slab separations occurred at all transverse joints during the first few days following construction. The observations in this respect are not considered to be of particular significance except possibly in connection with the different methods of curing used on the two sawed-joint sections.

As noted previously, behavior comparisons between wetted-burlap and impermeable-paper curing methods allied with the use of sawed joints were planned. Conclusive results of this research (curing methods in respect to sawed joints) failed to materialize because no random cracking occurred prior to or during sawing in either of two sawed-joint test sections. However, the time period between construction and slab separation was longer by several days on the wet-burlap-cured section than on any of the sections cured with impermeable paper.

No important differences in climatic conditions occurred during the period that the burlap-cured pavement was under observation, so it seems possible that the lack of early separation may have been due to the difference in curing. However, the evidence furnished by the present study is not sufficient to be considered conclusive.

Long-Term Slab Movements. –Long-term slab movements at joints and cracks have occurred in both vertical and horizontal directions. Averages of the vertical displacements measured at joints of the individual test sections during the first five years of pavement life are shown in Figure 6. It will be noted that average displacements are very slight thus far, although there is an apparent tendency toward an increase with age. The forward slabs in the direction of traffic are, on the average, becoming lower than the abutting slabs. No significant differences in the behavior of the individual test sections are apparent.

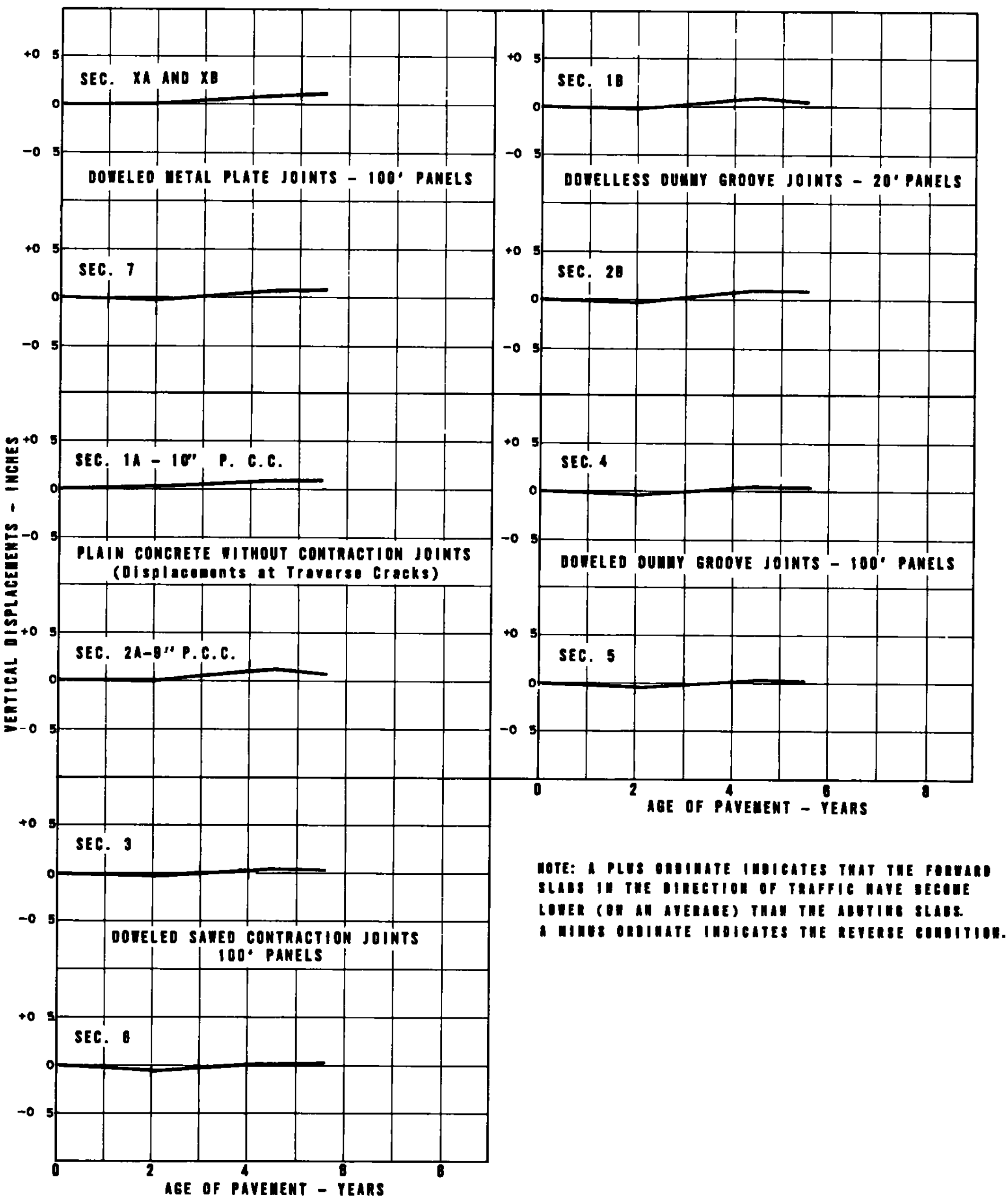


Figure 6. Vertical displacements of abutting ends of pavement slabs.

A conventional analysis of horizontal slab movement at joints and cracks is depicted in Figure 7. Although gaps in the field data reduce the value of the analysis, trends that were to be anticipated from the results of similar studies elsewhere are apparent. Opening and closure movements are least for the closely spaced joints (Sections 1B and 2B having 20-ft panels). Joints and cracks of all sections show a tendency to not return to complete closure. Joints spaced at 100-ft intervals maintain greater openings during hot weather than do the joints spaced at 20-ft intervals.

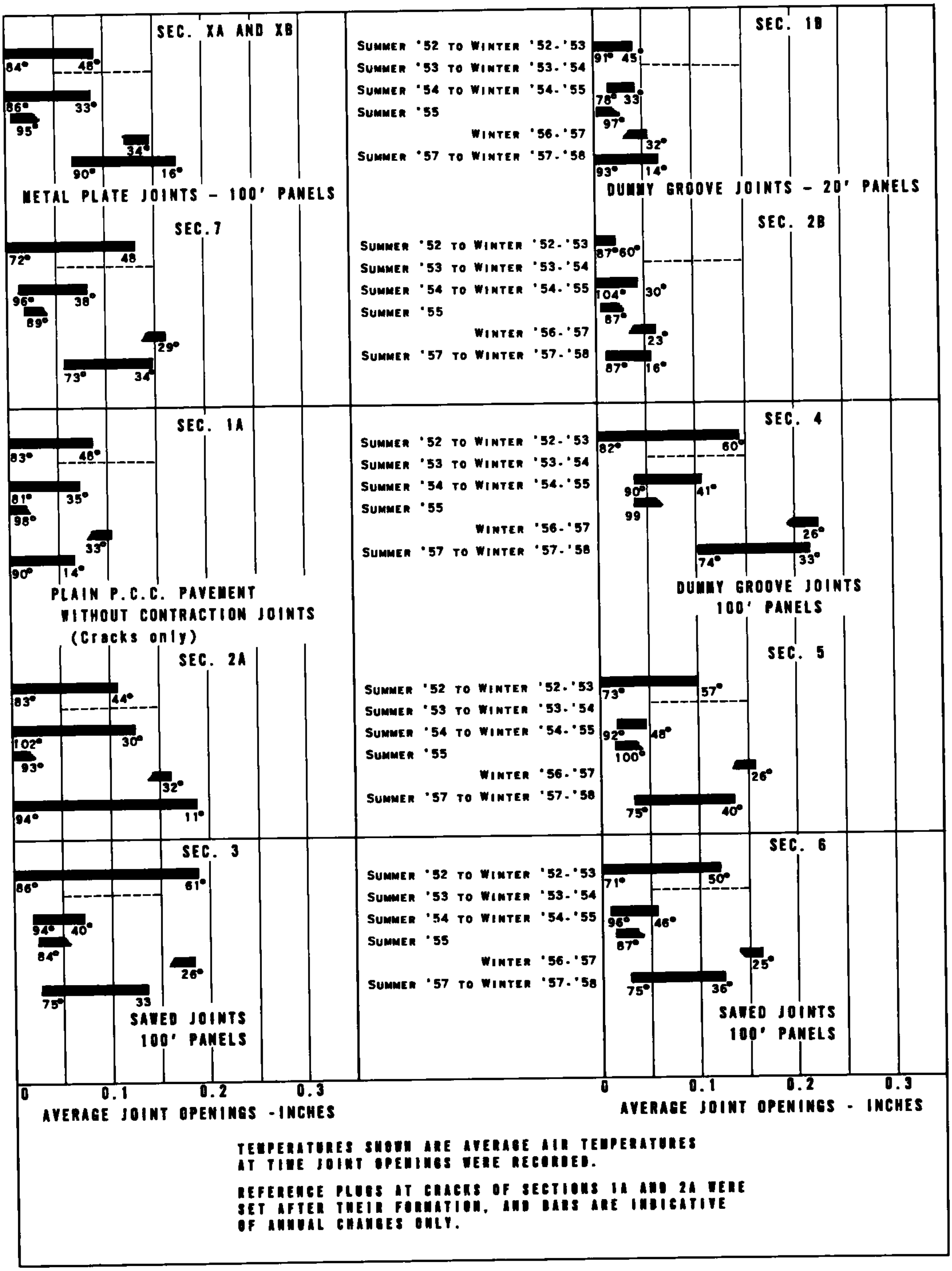


Figure 7. Annual and progressive changes in openings of transverse joints and cracks.

INTERIM FINDINGS

After seven years of service all of the experimental pavement is in acceptable condition, and such defects as have been noted are of a relatively minor nature. Nevertheless, some of the defects that have occurred have been concentrated on certain of the test sections and do offer a means of comparing the various experimental design features with respect to resistance to these defects. Pavement defects that have been noted thus far have been limited to transverse cracking, the formation of infiltration cracks, spall at joints and cracks, differential displacement of slabs at joints and cracks, and ineffectiveness of the various joints in the retention of seal. The riding quality of the various test sections as indicated by a road-smoothness indicator has also been determined.

Defects

Interim findings with respect to the various defects that have occurred are:

Transverse Cracking. –Transverse cracking is confined almost wholly to the plain unjointed test sections. A question arises as to whether the plain unjointed pavements should be compared adversely with the other pavements because of this concentration of transverse cracking in them. Inasmuch as the average uncracked slab length for the unjointed pavement is within the same range as the average uncracked slab length for the pavement with joints at 100-ft intervals, and is much greater than that for the pavement with joints at 20-ft intervals, it is considered that the unjointed pavements should not be looked on in an adverse light at the present time solely because of this cracking.

Infiltration Cracks. –Infiltration cracks are few in number and show no real pattern of occurrence with respect to the experimental designs. There is slight indication that they are more prevalent where the longer joint spacings were used.

Spall. –Spall, as would be expected, has occurred where edging was omitted at the full-depth metal-plate joints. Such spall is widespread but not of great depth or width. Spall is also much in evidence at the cracks in the unjointed pavements. This spall is also widespread and in a few instances quite severe with respect to depth and width. Very little spall has occurred elsewhere.

Faults. –Slab faulting is present at many of the joints and cracks. Differential displacements are slight, and not related to any particular joint design including the joints without mechanical load transfer devices. However, because the joints without load transfer devices are spaced at 20-ft intervals as compared with the 100-ft intervals of the joints with load transfer devices, the number of faulted slabs per unit length of pavement is greater for the sections containing these joints.

Riding Quality. –Measurements of riding quality made in 1957 and 1959 with a standard road-smoothness indicator showed the pavements jointed at 20-ft intervals to be inferior to the unjointed pavements and to the pavements with joints at 100-ft intervals. Other experimental features did not produce significant differences in riding quality as determined with the road-smoothness indicator. The roughness-index values of 70 to 90 in. per mile that were determined for the jointless pavements and for the pavements with joints at 100-ft intervals correspond with those determined with the same device for the majority of standard-design pavements with joints at 100-ft intervals for which roughness-index values have been determined in Illinois. The roughness values of 91 to 115 in. per mile for the pavements with joints at 20-ft intervals correspond with values determined for the rougher standard-design pavements in the state.

Shoulder Holes. –Pumping has not been noted on the test project. Shoulder holes without evidence of ejected material have been noted at the pavement edges near joints and cracks in a few instances, but their occurrence has been so infrequent and so random in location that little significance has been attached to them.

Condition of Seal. –Of two rubber-asphalt sealing materials which were used, one a hot-poured material and the other a cold-applied material, the hot-poured material showed somewhat better adhesion to the concrete and greater resistance to cracking and removal. Neither material was ever completely effective in cold weather under the conditions of this particular project.

The abrading of joint faces of hand-edged joints to remove surface mortar of the type that occasionally has been found pulled away from the main body of concrete by sealing materials caused some undesirable spall and did not produce uniformly abraded faces with the methods and equipment which were used. There was no significant evidence to indicate that the sealing material adhered to the concrete better at the abraded joints.

Experimental Features

In consideration of the distribution of the various defects that have occurred, interim findings with respect to the experimental features are:

Pavement Design.—The best service is being rendered currently by Illinois standard steel-reinforced pavement with joints at 100-ft intervals and load-transfer devices at the joints. The jointless pavements without steel reinforcement, and the nonreinforced pavements with joints at 20-ft intervals and no load transfer devices are rendering satisfactory service, though not in such a high degree as the standard pavement. The jointless pavements of 9-in and 10-in. thickness are of approximately equal behavior after seven years of service, as are also the 9-in. and 10-in. thick pavements with joints at 20-ft intervals.

Contraction Joints.—Little difference has been noted thus far in the behavior of the impressed joints with hand-tooled edges, the standard metal-plate joints with hand-tooled edges, and the sawed joints. Non-edged metal-plate joints show considerable slight spall and poor retention of seal. Joints with and without load-transfer devices show little difference in behavior thus far; however, the differences in the joint spacing of these two designs must be considered as a probable factor when making this comparison.

Sealing Compounds.—Under the conditions of this particular project, the hot-poured rubber-asphalt sealing compound has shown somewhat better adhesence to the concrete and less tendency toward raveling and rupturing than has the cold-applied rubber-asphalt seal. Neither type of seal has remained unbroken during cold weather. At seven years neither material appeared to be functioning and all joints were resealed.

Abrasion of Joint Faces.—The device used for abrading joint faces for the purpose of removing any weak mortar film that might have accumulated did not provide continuous abrasion. There was no significant evidence to indicate that the abrasion process as used on this project was truly effective.

Curing.—In the study designed to determine the influence of impermeable-paper curing and wetted-burlap curing on premature cracking where sawed joints are used, no cracks developed on either test section. However, some observations of slab separation at the transverse joints indicated that there was less movement during the first few days after construction where the wetted-burlap cure was used.

Colorado Concrete Pavement and Subbase Experimental Project

CHARLES LOWRIE and W.J. NOWLEN, respectively, Assistant Materials Engineer, Colorado Department of Highways; and Associate Development Engineer, Portland Cement Association

In 1952 the first two lanes of a planned four-land concrete highway between Denver and Castle Rock, Colo. were placed. The alignment for the highway traversed areas having soil of high swell characteristics. Within a short time after placement, the pavement warped and cracked in certain areas. In 1956 the additional two lanes were placed. In this construction, the subgrade was adjusted on the wet side of optimum moisture for the specified compactive effort. Five pavement structures were incorporated in the job: an 8-in. plain concrete slab built on a 4-in. granular subbase treated with 2 percent portland cement, a 10-in. plain pavement on a 4-in. subbase, an 8-in. plain pavement on a 20-in. subbase, and two mesh-reinforced 8-in. pavements on a 4-in. subbase. The joint spacing was 106 ft in one reinforced section and 61 ft in the other.

During the spring of 1959 the five experimental designs were tested under dynamic and static wheel loads. Deflections and strains due to dynamic loads and deflections under static loads were measured. The modulus of subgrade reaction was determined from plate bearing tests.

Data from the tests show that the reinforced pavement designs having a relatively long joint spacing undergo greater corner deflections under load, and attain higher curl than the plain concrete pavement having relatively short joint spacing. An experimental section having a 20-in. cement-treated subbase gave lowest corner deflection.

Only limited conclusions are drawn pending further visual observations and testing after a longer period of service.

● THE GRADING of the first two lanes of a planned four-lane highway between Denver and Castle Rock, on US 87, was completed during the years 1950 and 1951. The alingment for this new highway traversed areas having soils of high-swell characteristics. Specifications for grading and compaction required a minimum density of 90 percent of AASHO modified density (1).

In 1952 pavement was placed on the completed subgrade. This consisted of a 6-in.

layer of granular subbase material and an 8-in. uniform thickness portland cement concrete pavement. Shortly after the pavement was placed, cracking and warping of slabs was observed in certain areas. It was generally believed that the amount of water used during grading operations was not sufficient to allow the swelling soils to reach equilibrium before the pavement was placed.

The grading work for the additional two lanes was placed under contract in 1956. Specifications for this grading required 95 percent of AASHO T 99 standard density (2). This project was constructed slightly on the wet side of optimum moisture for the specified compactive effort. It was believed that the swell potential of the swelling soils had thereby been largely reduced. A pavement design was chosen as shown in Figure 1. The granular subbase material (4 in. thick) was treated with 2 percent portland cement to produce a minimum stabilometer resistance value of 80, not a hardened soil-cement material. An 8-in. uniform thickness portland cement concrete pavement was considered adequate. This design is referred to later as the "typical section."

It was realized that this pavement project afforded an excellent opportunity to compare the performance of similar subgrade soils compacted under AASHO T180 modified compaction and under AASHO T99 standard compaction. At the same time, it was considered advisable to include some variables in pavement design. If the new pavement should remain smooth, it would be assumed that sufficient moisture had been added to prevent swelling of the subgrade soils and the pavement variables might not provide any pertinent information. On the other hand, if distortion and cracking should occur, the pavement variables might provide some information regarding their relative merits. Later, it developed that there would be an opportunity to load-test the various pavement sections.

PAVEMENT DESIGN VARIABLES

Five experimental pavement-subbase designs were included in the project, as follows:

Section A (Station 377+49 to 403+96)—2,647 linear feet of 8-in. concrete pavement reinforced with 6 x 12¼ welded wire fabric (weighing 61 lb per 100 sq ft), on 4-in. cement-treated subbase. Fabric placed 2 in. below pavement surface. Sawn contraction joints spaced at 61 ft, 6-in. centers. Dowels: 1-in. round, spaced at 12-in. centers.

Section B (Station 403+96 to 429+99)—2,603 linear ft of 8-in. concrete pavement, reinforced with 6 x 12⁰⁰/4 welded wire fabric (weighing 79 lb per 100 sq ft), on a 4-in. cement-treated subbase. Fabric placed 2 in. below surface of pavement. Sawn contraction joints spaced at 106 ft, 6-in. centers. Dowels: 1-in. round, spaced at 12-in. centers.

Section C (Station 573+00 to 625+83.6)—5,283.6 linear ft of "typical section": 8-in. plain concrete pavement on 4-in. cement-treated subbase, sawn contraction joints at 20-ft centers. No dowels.

Section D (Station 625+83.6 to 652+23.6)—2,640 linear ft of 10-in. plain concrete

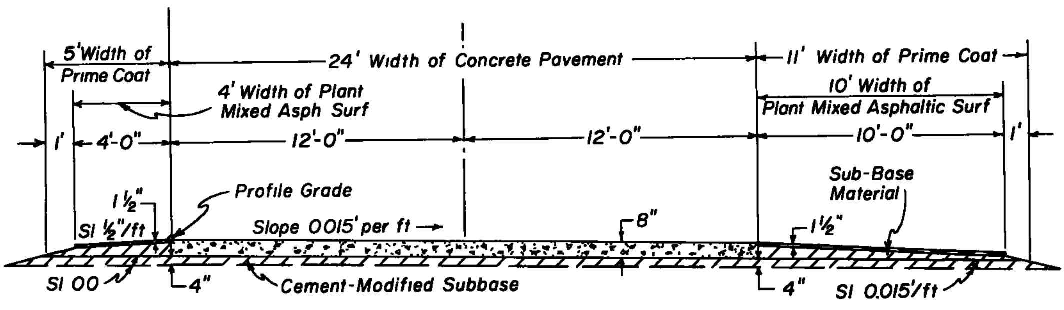


Figure 1. Typical section.

pavement on 4-in. cement-treated subbase. Sawn contraction joints spaced at 20-ft centers. No dowels.

Section E (Station 656+00 to 680+00–2,400 linear ft of 8-in. plain concrete pavement on 20-in. cement-treated subbase. Sawn contraction joints spaced at 20-ft centers. No dowels.

MATERIALS AND CONSTRUCTION

The predominant subgrade soils in the top 3 ft of all five experimental pavement sections are within the range of A-7-6 (12) to A-7-6 (20).

A typical grading of untreated material for the cement-treated subbase is as follows:

Passing 3/8 in.	100%
Passing No. 4	99%
Passing No. 10	90%
Passing No. 40	52%
Passing No. 100	26%
Passing No. 200	22%
Plasticity Index	8

After treatment with 2 percent portland cement, Atterberg tests showed the subbase material to be non-plastic.

Aggregate for the 8-in. concrete pavement had a maximum size of 2 in. The cement factor was 6 sk per cu yd and the average 28-day compressive strength of test cylinders was 4,244 psi. Entrained air in the fresh concrete was maintained between 4 and 5 percent.

The pavement was placed by means of a slipform paver (3). This necessitated the use of concrete having a slump of about 1¾ in. to 2 in. For placement of the mesh reinforcement, an improvised frame was attached to the front of the slipform paver. As the frame was pushed ahead of the paver, it held the wire mesh off the prepared subbase at the required height of 6 in. (2 in. below the finished surface of the pavement).

Although no traffic count has been made at the exact location of the test sections, counts made in 1958 (year of completion of pavement) at junctions about 10 mi north and south, indicate the average daily vehicle (ADV) count to be 6,000 for the 4-lane facility. However, the completion of a connecting freeway through Denver may have increased the ADV by as much as 20 percent.

After completion of paving operations, profile elevations were taken on all the test sections. Elevations were taken again approximately 1 yr later. Although some uplift has been observed, trends are not sufficiently developed to warrant a conclusion at this time. Pavement distortions that have occurred on the five test sections are not nearly as severe or widespread as those which have occurred on the opposite two lanes placed under drier soil conditions in 1952.

As an additional check on pavement distortion, an as-completed profile of the pavement was made during the summer of 1958 using a California profilometer. It is expected that similar observations to be made in the future will trace the changes in pavement profile.

LOAD TESTING PROGRAM

The load testing program was carried out during the spring of 1959 as a cooperative project of the Colorado Department of Highways and the Portland Cement Association.

Static and moving load tests were made, and strains in the concrete as well as deflections of the pavement were measured. These data permitted a comparison of the 5 experimental pavement designs. The instrumentation installed for measuring deflection was semi-permanent, and it is anticipated that additional tests will be made in 2 or 3 years.

A 4-axle semitrailer and tractor combination (Fig. 2) was used for loading, and measurements of pavement performance were made by maximum-reading deflectome-

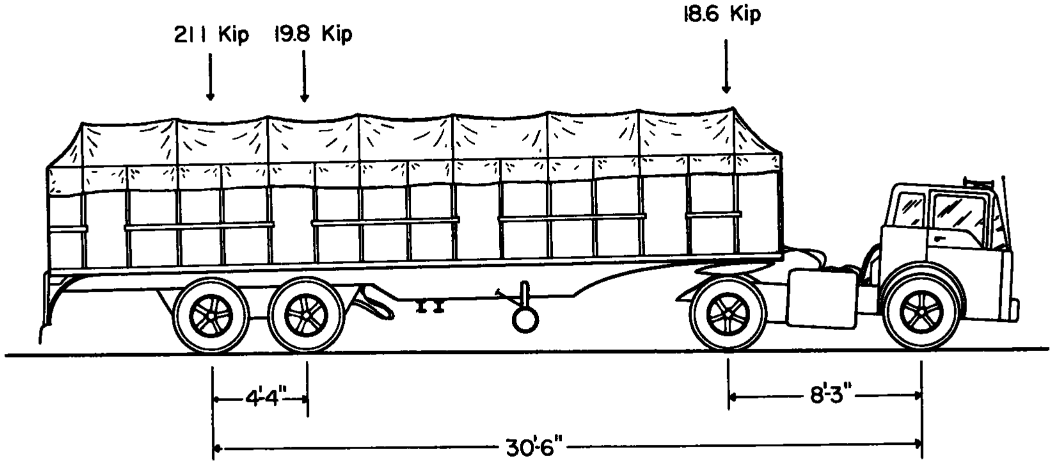


Figure 2.

ters and A-9 strain gages. Details of the loading and instrumentation are described later. Temperatures of the pavement surface, of the interface of the pavement and subbase, and of the air (shaded and unshaded) were registered on a continuous recorder.

Plate bearing tests were made on the subbase. Compression tests and static modulus of elasticity tests were made on 6-in. cores removed from each pavement section.

Three test sites were chosen from each of the experimental sections A through E. Each test site included four successive slabs which were instrumented as shown in Figure 3. Moving loads were applied to all of the sites by moving the loading truck in two main loops, each approximately 6 mi long. One loop included the test sites

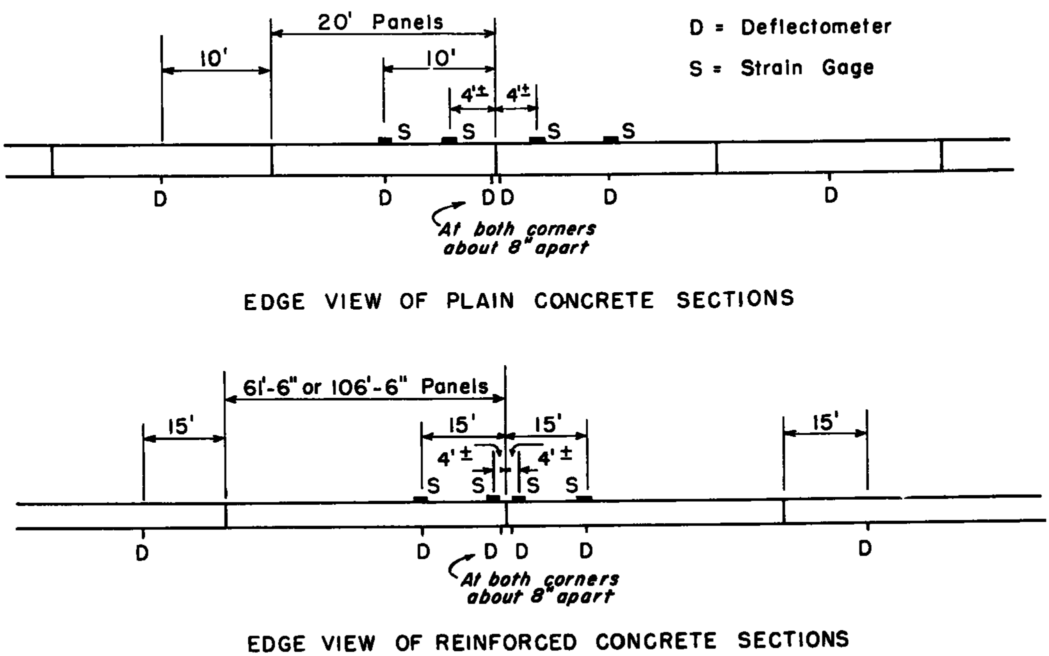


Figure 3. Location of deflectometers and strain gages

in sections A and B; the other included the test sites in sections C, D, and E. Loading tests were made three times during the day: between 5:30 a.m. and 7:00 a.m., between 10:30 a.m. and 12:00 noon, and between 3:00 p.m. and 4:30 p.m. Each loading test consisted of at least 3 trips of the truck over the instrumented slabs.

INSTRUMENTATION AND TEST PROCEDURE

The semitrailer truck was loaded with sand ballast so that the single-axle load was 18,600 lb and the tandem-axle load was 40,900 lb, of which 19,800 lb was on the forward tendem axle and 21,100 lb on the rear tandem axle (Fig. 2). Tire pressures were maintained at 80 psi. All load tests were made with the wheels of the truck running within 2 to 3 in. of the edge of the pavement at 2 to 3 mph. Outside wheel loads due to regular traffic normally would be about 30 in. from the pavement edge. However, occasional loadings nearer the edge are inevitable and would represent more severe loading conditions. These tests, intended to form the basis of a design comparison, were made near the edge so that design differences would be maximized.

The type of maximum-reading deflectometer used is shown in Figure 4. The "sensing element" consists of a vertical steel pin held in a machined brass housing by a leather friction pad. The housing is attached to an adjustable bracket which is in turn attached to the vertical edge of the concrete slab in such a way the deflectometer pin rests on an 8-ft bench rod driven through a 5-ft long casing into the subgrade. During passage of the test load, the pin is displaced vertically relative to the housing. The displacement corresponding to maximum deflection of the pavement is retained, and is measured later by a 0.001-in. dial indicator equipped with an adapter. During

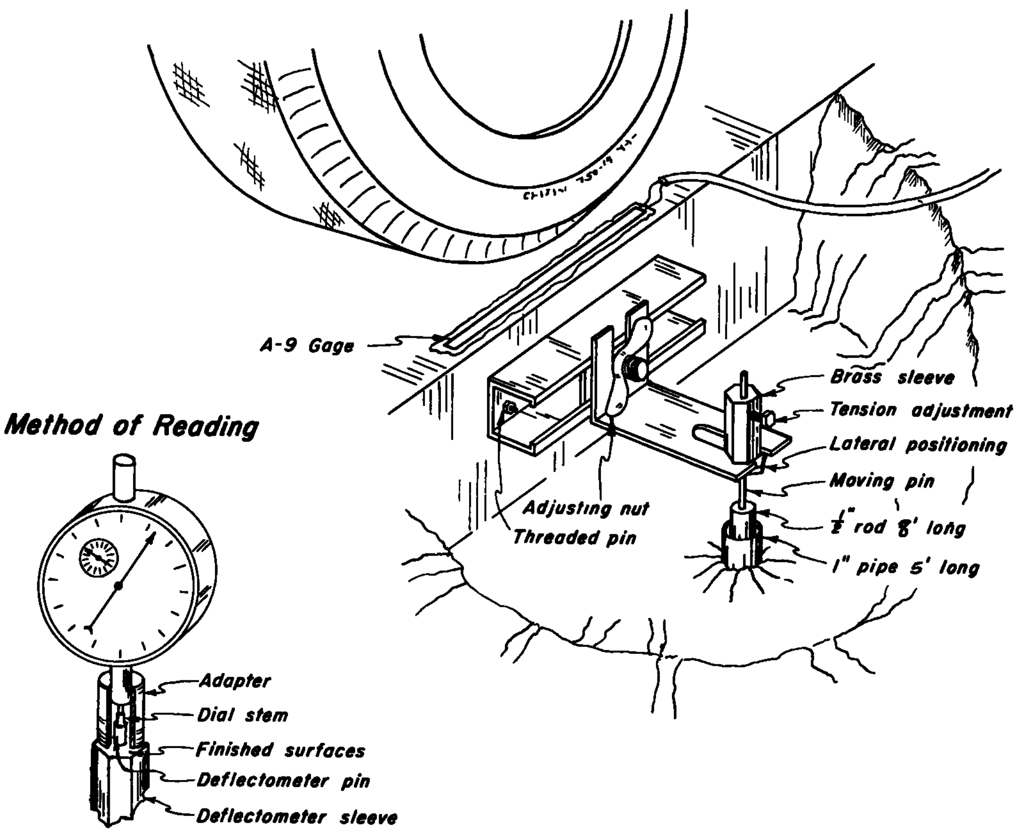


Figure 4. Deflectometer installation.

the day the change in pin position of the unloaded slab was also used for measuring the change in curl of the slab.

Strain gages were bonded to the surface of the pavement near the edge and were waterproofed with a tough wax. Changes in strain as the truck wheels approached and passed each instrumented point were recorded as traces on a strain indicator housed in an instrumentation trailer.

Plate bearing tests on the subbase were made by applying loads through a 30-in. steel plate 1 in. thick using the loading truck as a reaction for the hydraulic ram. For each test an excavation was made in the shoulder material to a level corresponding to the bottom of the pavement. The plate was placed on a thin layer of plaster of Paris and leveled. Four dial indicators were used to measure the downward motion of the plate. The dials were attached to wooden bridges supported at the pavement edge on one side and extending laterally to a point on the shoulder about 10 ft from the pavement edge.

FIELD TEST RESULTS

Temperature and Curl

The load-deflection characteristics, and to some extent the load-strain characteristics, of a concrete pavement are dependent on the extent to which the slab is curled upward or downward at the edges and the corners. Daily changes in curl are due principally to the differences in temperature between the top and bottom of the slab. Moisture content differences also contribute to curl but mostly on a "long-time" basis rather than a day-to-day basis (excluding the effects of rain).

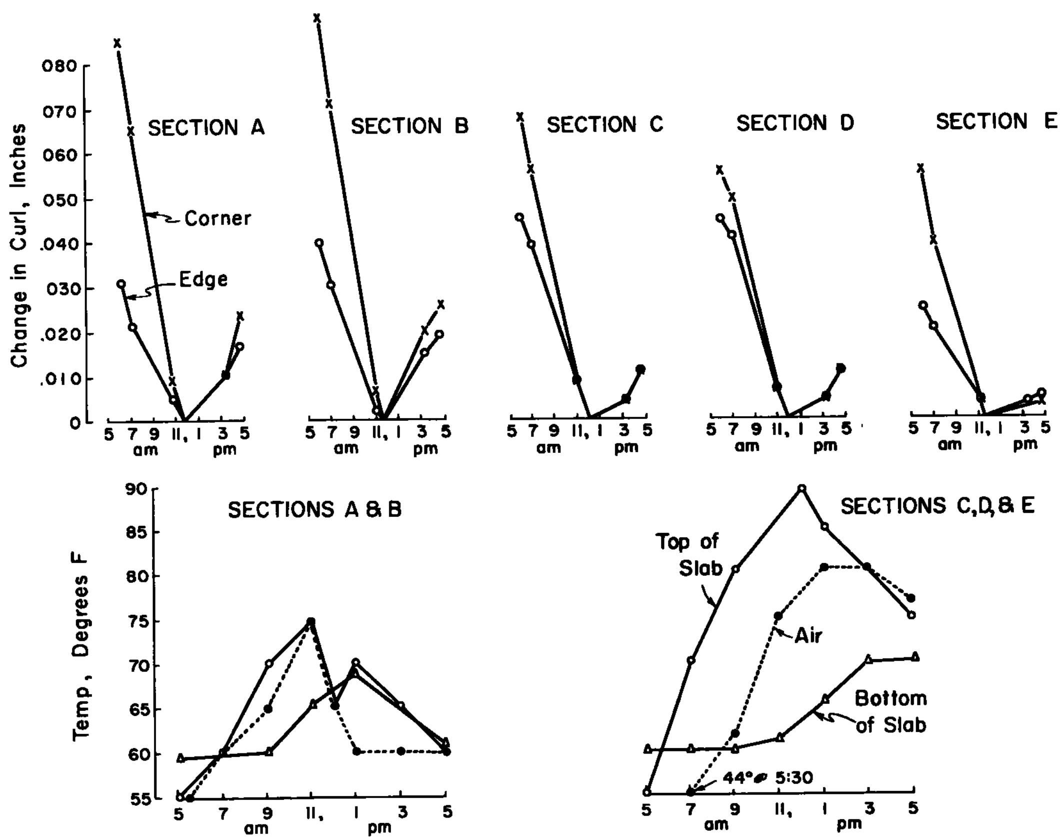


Figure 5. Temperature and curl vs time.

The changes in temperature of the surface of the slab and of the bottom of the slab for test installations in each of the 5 sections are shown in the lower part of Figure 5, and the changes in curl with respect to temperature are shown in the upper part of the figure. The plotted points for "change in curl" represent the average of 6 corner measurements and 12 edge measurements from each section. The change in curl was very rapid during the early morning hours when the temperature of the top of the slab was increasing rapidly. During the middle of the day all slabs ceased their downward curling and began to curl in the opposite direction.

The change in curl at the corners of sections A and B (long joint spacing) was greater than in the other sections tested even though a rapid drop in temperature of the top of the slabs occurred at 11:00 a.m. due to a brief rain shower.

Deflections–Moving Load Tests

In general, the deflection results reflected fairly consistent behavior of the test slabs in any one section. The data from at least 3 test runs over the slabs in each of the three sites of a section were averaged to permit a comparison of the structural behavior of the pavement in the 5 test sections. These deflection data are given in Table 1 for corner and edge positions. It will be recalled that these are maximum deflections registered by the passage of a loaded truck having a 40,900-lb tandem-axle load. The early morning corner deflections of reinforced sections A and B with the longer joint spacing are higher than those of the other sections. Corner deflections were lowest in section E regardless of time of day.

TABLE 1

AVERAGE OF MEASURED MOVING LOAD DEFLECTIONS (0.001 in.)

	Corner					Edge				
	Sections					Sections				
	A	B	C	D	E	A	B	C	D	E
Early a.m.	47	50	24	19	19	26	22	22	17	11
Late a.m.	17	11	8	11	7	11	10	9	11	11
Late p.m.	13	13	7	13	7	13	12	8	14	9

It may be seen from Figure 5 that the temperature conditions existing when sections A and B were tested differed from those existing during the testing of C, D, and E. Therefore, the data in Table 1 for the different experimental sections are not directly comparable. This is particularly true of the readings made in the early morning when it is known that the degree of upward curl of the pavement, and hence the deflections under load, can vary significantly on different days. Late morning and early afternoon readings may be compared with less question.

Deflections–Static Load Tests

Static wheel tests were made by placing each wheel of the truck at the edge of the slab at successive instrumented points. Measurements of deflection were made on each side of the joint for each wheel position. Table 2 gives measured deflections on either side of a joint when the load is on the approach slab, together with edge deflections. No great differences are apparent in corner deflections of sections A, C, and D. However, the deflections of section E, are significantly lower than those of the other sections and the deflections of section B appear to be significantly greater. This may be explained by the fact that section E has a 20-in. cement-treated subbase which may reduce deflections, and section B has the longest joint spacing which results in wider joint openings and more

TABLE 2

STATIC WHEEL LOAD DEFLECTIONS
(0.001 in.)

Time	Section A				Section B				Section C				Section D				Section E			
	Single		Tandem		Single		Tandem		Single		Tandem		Single		Tandem		Single		Tandem	
	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.
(a) Corner																				
a.m.	15	14	20	18	30	25	37	26	13	10	18	14	20	20	28	27	6	5	7	7
p.m.	12	12	18	16	14	14	20	18	8	8	10	8	14	15	23	22	5	5	7	7
(b) Edge																				
a.m.	10		16		13		21		9		17		19		31		6		10	
p.m.	11		17		9		16		7		13		12		25		9		11	

Ap. = Slab on which truck approaches joint (loaded slab).
Lv. = Slab on which truck leaves joint (unloaded slab).

curl which may reflect in higher corner deflections. It is unusual that the deflections of section D, the 10-in. thick pavement, are just as great as those of the 8-in. pavements.

The amount of load transfer attributed to aggregate interlock and to dowels is reflected by the difference in deflection of one side with respect to the other. Joint "effectiveness" was computed by the method of Teller and Sutherland (4), using

$$E_1 = \frac{2\, d_j'}{d_j + d_j'} \tag{1}$$

in which d_j' is the deflection of the unloaded joint-edge and d_j is the deflection of the loaded joint-edge. Values computed by using the expression are given in Table 3.

TABLE 3

JOINT EFFECTIVENESS
(Computed from Data in Table 2(a))

	Section A		Section B		Section C		Section D		Section E	
	S	T	S	T	S	T	S	T	S	T
a.m.	0.97	0.95	0.91	0.83	0.87	0.88	1.0	0.98	0.91	1.0
p.m.	1.0	0.94	1.0	0.95	1.0	0.89	1.0	0.98	1.0	1.0

S = effectiveness under single-axle loading.
T = effectiveness under tandem-axle loading.

The "joint-effectiveness" ratio is relatively high for all sections; however, sections D and E show some advantage over the other sections. It should be mentioned that the joint-effectiveness ratio will undoubtedly change during the year, and that in cold weather, when the slabs are in a contracted condition, aggregate interlock may be reduced in sections C, D, and E. This would reflect in a lower joint-effectiveness ratio.

Pavement Strains

Strains were measured along the edge of the slabs at two sites in each of the 5 sections. Gages were placed at 4 ft and 10 ft from the corner in the short slabs of sections C, D, and E, and at 4 ft and 15 ft in the longer slabs of A and B sections (Fig. 2). Averages of the strain readings for each section are given in Table 4. Strains were measured at creep speed and were recorded as a continuous trace (Fig. 6). Although both compressive and tensile strains were recorded, only the compressive strains in the top of the slab occurring as the wheels moved to a position adjacent to the gage are given in Table 4, because these were the larger strains. It has been

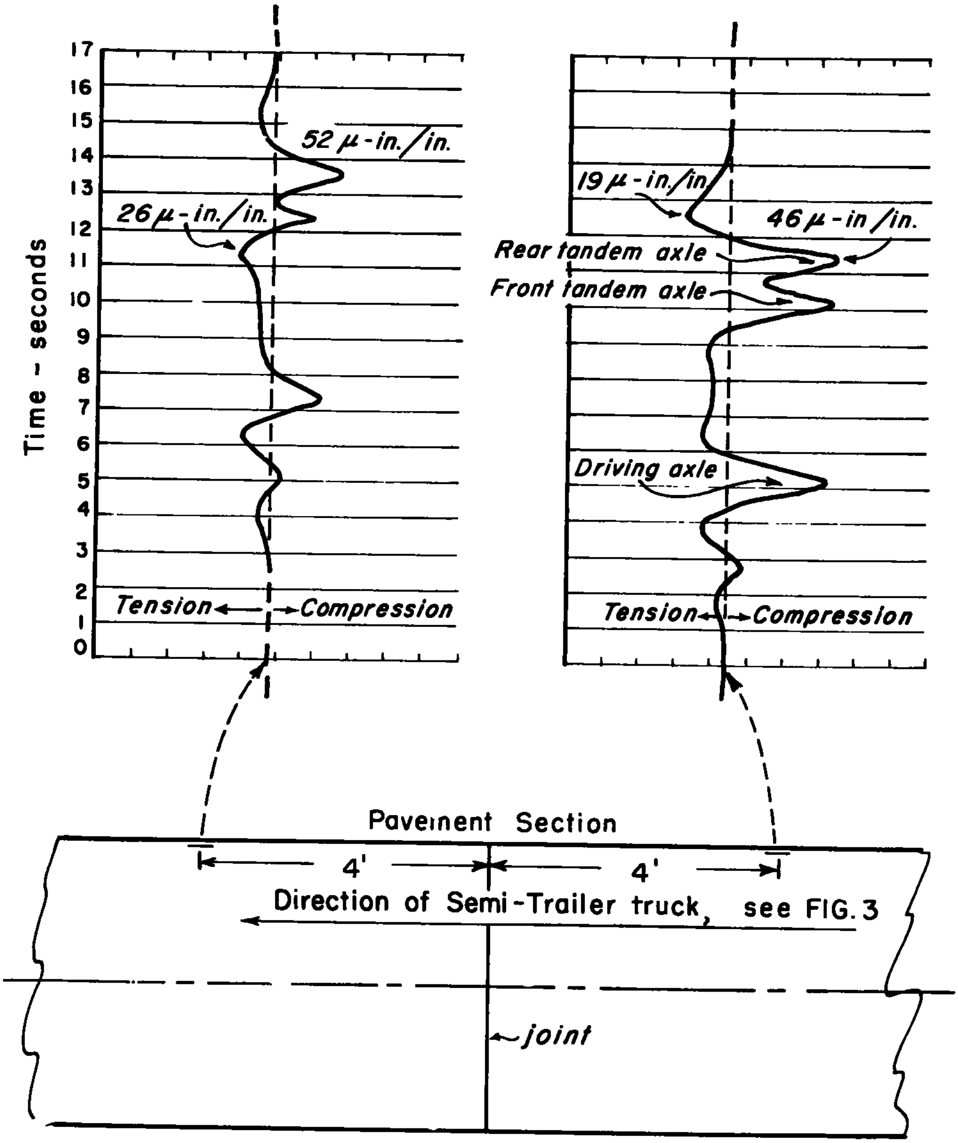


Figure 6. Strains near joint semi-trailer truck moving along slab edge.

TABLE 4

AVERAGE STRAINS
(Millionths)

		Section A			Section B			Section C			Section D			Section E		
		S	FT	RT	S	FT	RT	S	FT	RT	S	FT	RT	S	FT	RT
A.M., 4 ft from joint	AP	51	36	54	52	48	44	44	28	42	47	29	49	40	33	50
	LV	43	34	49	41	23	44	48	29	46	33	26	31	40	30	34
A.M., 10 or 15 ft from joint	AP	29	25	35	55	41	58	43	30	43	41	28	44	37	28	40
	LV	32	27	43	33	25	39	54	36	55	37	29	38	33	25	35
P.M., 4 ft from joint	AP	53	50	61				45	32	39	49	40	52	39	27	41
	LV	39	26	49				46	34	44	45	31	45	43	28	42
P.M., 10 or 15 ft from joint	AP	30	28	39				40	27	40	45	35	42	43	39	48
	LV	19	15	22				41	28	40	41	39	44	43	27	44

S = Single driving axle.
FT = Front axle of tandem.
RT = Rear axle of tandem.
AP = Slab on which truck approaches joint.
LV = Slab on which truck leaves joint.

anticipated that the tensile strains near a joint, when the load was at the joint, might be critical in some cases. However, as load transfer was good, tensile strains at this location were never very great. The strains measured in section A, 15 ft from a joint, were unusually low as compared with strains in other sections. No reason for this is readily apparent.

Strains of the order measured in these tests developed stresses ranging from 50 psi to 200 psi. Stresses in this magnitude should have no harmful structural effect on the pavement.

Supplementary Tests

Curves showing the load-deflection characteristics of the subbase-subgrade are shown in Figure 7. Values obtained at section C are believed to be influenced greatly by a local shale deposit near the surface at the test site. Results for sections B and E are in the order of those to be expected. The 20-in. layer of cement-modified soil in section E resulted in a very high k value of 600 pci when computed by a simple formula of unit load on the plate divided by the average plate deflection at that load.

Representative values for the compressive strength, the static modulus of elasticity in compression, and Poisson's ratio were obtained from compression testing a 6-in. core from each of sections A, C, D, and E. These data are given in Table 5.

CONCLUSIONS

It is believed that tests of the type described in this paper provide significant information as to the probable future behavior of a concrete pavement. Although only limited conclusions may be drawn from this one load-testing program, it is not unreasonable to believe that correlation of these tests with duplicate tests planned for 2 to 3 yr hence, plus interim visual observations, may provide a behavior pattern by which other roadways of this type may be evaluated early in their life.

Insofar as deflections and strains are concerned, all values were relatively low. Therefore, differences observed in the behavior of the various experimental sections are not considered significant at this time. The 8-in. plain concrete design of section C compared favorably with the 8-in. reinforced designs of sections A and B. Section

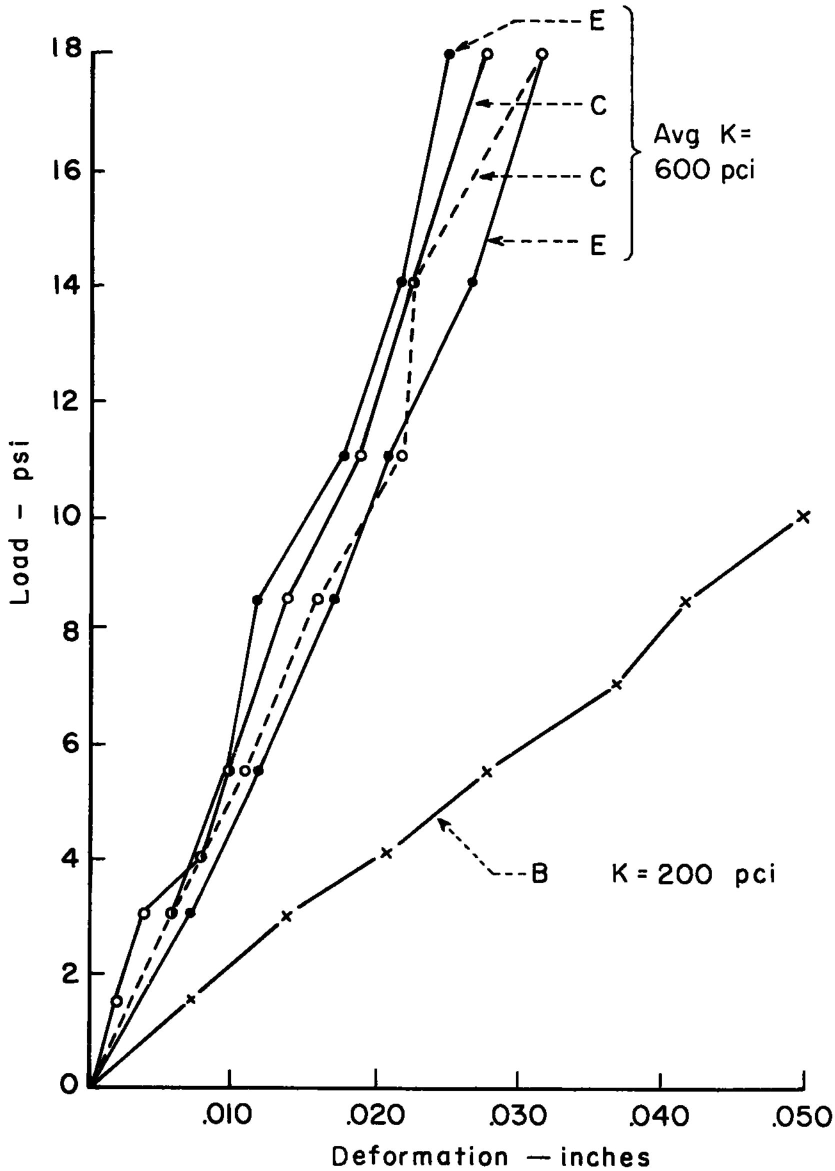


Figure 7. Plate bearing tests load vs deformation.

TABLE 5

COMPRESSION TEST RESULTS-6-IN. CORES TESTED DRY

Section	Compressive Strength, psi	Static E Millions psi	Poisson's Ratio
A	5,500	3.8	0.22
C	5,300	3.8	0.18
D	4,700	3.4	0.17
E	5,400	3.8	0.21

E, which included a 20-in. cement-treated subbase, generally had the lowest deflections and strains. Sections A and B had the highest corner and edge curl, and section E had the lowest edge curl under temperature conditions existing at the time of test.

Temperature variations are, of course, always present, and have a significant effect on the experimental data. Therefore, any quantitative comparisons between sections tested at different times must be considered in the light of such variations.

It should not be inferred that deflections such as measured during the early morning hours in sections A and B are detrimental unless it is shown that the pavement cannot sustain these deflections over a long period of time. Neither can it be inferred that section E is the best by virtue of lower deflections and strains because other factors in the future may contribute to performance.

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HRB:OR-390

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