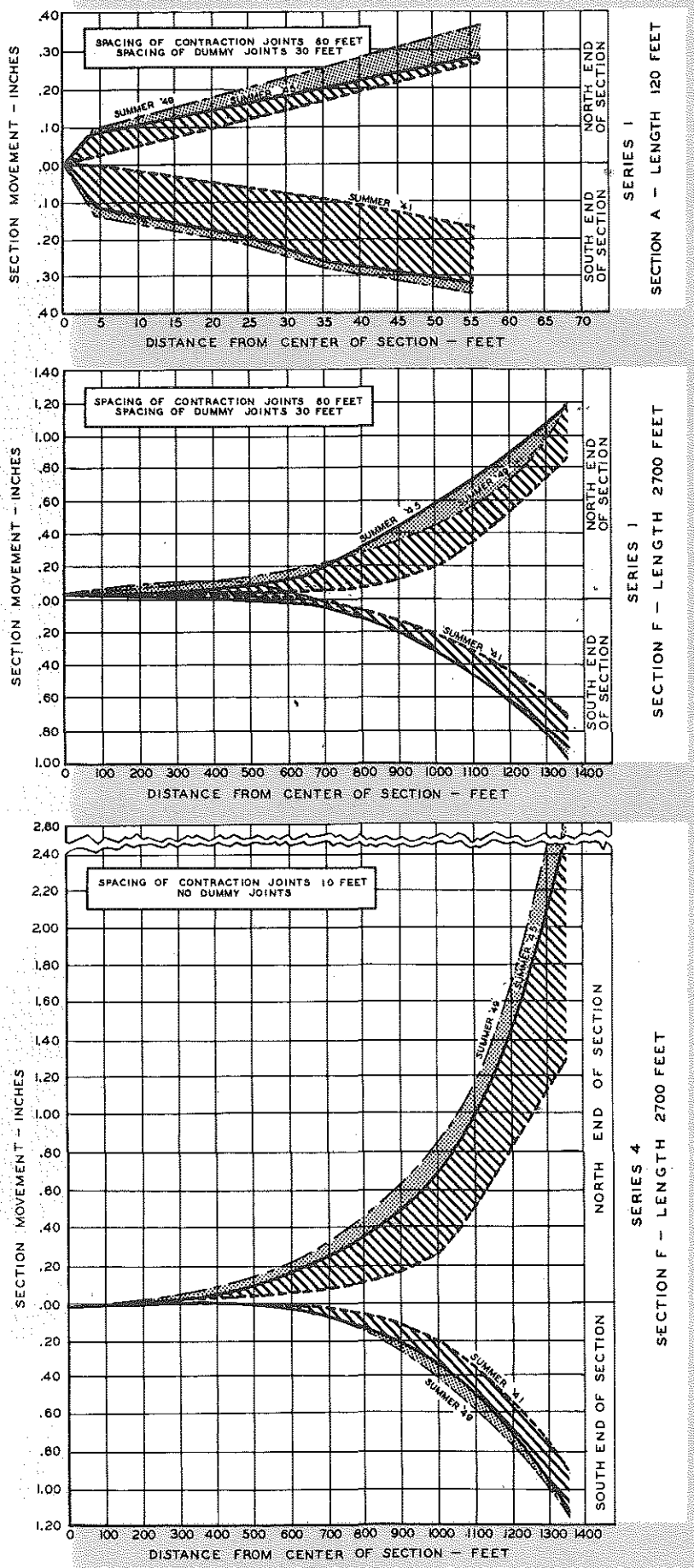


Figure 24

RELATION BETWEEN SECTION MOVEMENT AND DISTANCE FROM CENTER OF SECTION

to that of a continuous slab without expansion joints. Therefore, in the case of sections whose lengths are less than about 1800 feet, it may be expected that every point in each half of the section will display some movement with respect to the center of the section. For short sections such as illustrated by the graph at the top of Figure 24 (Series 1A) the movement of any point in either half of the section is approximately proportional to its distance from the center of the section.

It may be noted further that movements of the ends of each section are quite similar in character. In some instances certain inherent construction features, such as horizontal or vertical alignment, soil conditions, and bridge structures, no doubt influence the relative movement of the entire section, resulting in a general displacement of the whole section toward the right or left causing the point of zero movement to occur on either side of the geometric center of the section. It is also indicated in Figure 24 that the sections experienced their greatest movement during the first 5 years after construction.

SUMMARY

The study of expansion and contraction joint movement has brought out several interesting and significant facts concerning slab behavior under varying expansion and contraction joint spacing. *First,* the seasonal movements of the expansion joints indicate that there takes place during the first year after construction a considerable expansion and subsequent permanent displacement of the slab ends, using at least 50 per cent of the space originally provided. *Second,* subsequent to the first year's movement the section ends oscillate with seasonal climatic changes and the amplitude of these seasonal movements gradually diminishes with time. A slow, progressive permanent displacement also takes place which is greatest during the first 5 to 6 years and levels off thereafter. Eventually the joint filler will become compressed to such a state that no further longitudinal movement can occur. *Third,* contraction joint spacing has considerable influence upon the amplitude of expansion joint movement. *Fourth,* all contraction joints acquire a small permanent opening which increases with time. The degree of joint movement and amount of residual opening is more pronounced as the distance between contraction joints is increased. *Fifth,* the movement of contraction joints is greater near the expansion joints than it is near the center of the sections. *Sixth,* dummy joints react similarly to contraction joints but to a much smaller degree. *Seventh,* in sections of pavement greater than 1800 feet in length without expansion joints, there is a point of zero longitudinal extension approximately 700 to 900 feet from the ends of the section. Consequently, the central portion of such sections at elevated temperatures will be under restraint similar to that of continuous slabs in which no expansion joints have been provided.

EXPANSION AND CONTRACTION JOINT DESIGN

The design of a transverse joint necessitates consideration of structural features which will enable the joint to perform the function for which it is intended. It must provide for movement due to expansion or contraction, load transfer where necessary to maintain vertical alignment of abutting slabs, possess flexibility to permit warping, and provide adequate seal against infiltration of water and inert material.

Several units of various types of expansion and contraction joint construction in current practice were installed in the Design Project of the Test Road for comparative study. With respect to expansion joint construction two types of construction features were given major consideration; *first,* the efficacy of standard dowel bar construction with fiber filler strip versus air chamber construction, and *second,* different design features to maintain vertical alignment of abutting slabs. In the case of contraction joints special consideration has been given to the study of four outstanding construction features: *first,* the sealed groove versus premolded bituminous fiber strip; *second,* load transfer feature to maintain vertical alignment of the slabs; *third,* features to insure proper formation of contraction joint; and *fourth.* the feasibility of omitting load transfer devices in the case of short slabs and long spacing of expansion joints.

EXPANSION JOINT DESIGN

The following types of expansion joint construction have been considered:

Type DB-1 (1)	Highway Department Standard one-inch bituminous fiber board with ¾- by 15-inch dowel bars at 15-inch spacing for load transfer.
Type TE (1)	Thickened edge slabs with one-inch bituminous fiber board and one 1¼- by 18-inch dowel placed at each of the four corners, 9 inches in from the slab edge.
Type CB (2)	Uniform thickness slabs with one-inch bituminous fiber board and one 1¼- by 18-inch dowel placed at each of the four corners, 9 inches in from slab edge.
Type A (2)	Standard one-inch bituminous fiber board, but with no load transfer device.

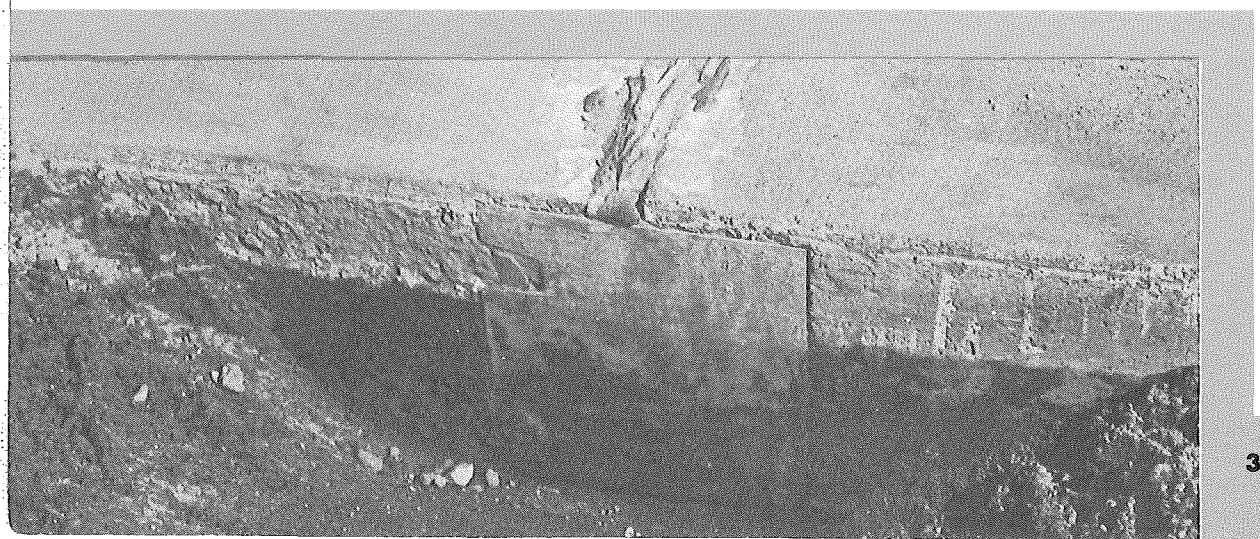


Figure 25A

Air chamber expansion joint Type DB-1(3), asphalt-latex seal, Series 5, at time of construction (1940), Station 79 + 60.

Type DB-1 (3) Air-chamber construction with one-inch opening, top, bottom, and sides sealed with an asphalt-latex joint seal compound and using ¾- by 15-inch dowel bars at 15-inch spacing for load transfer.

Type TB (4) Air-chamber construction with one-inch opening sealed at bottom with an asphalt-latex joint seal compound and at top and sides with a premolded rubber seal. Translode base units were used as the load transfer feature.

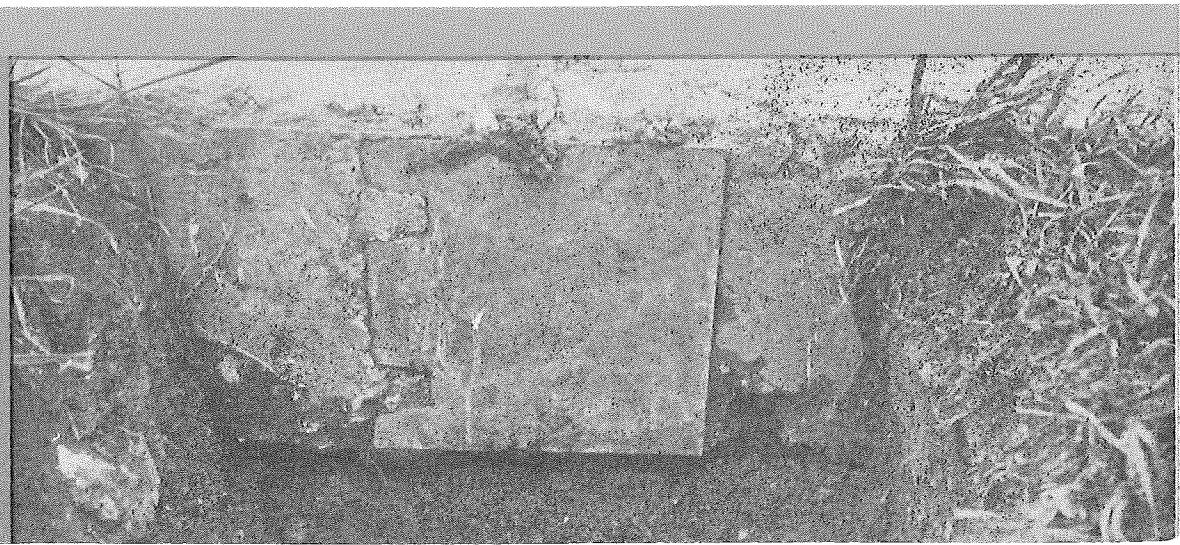
At the present time nothing has developed in the pavement structure nor shown up in the movement data which would indicate that any of the various types of expansion joints are not behaving in a normal manner as far as slab movement is concerned. Data presented previously in the report show that with few exceptions, such as in the stress curing section, the seasonal amplitudes of joint movement are comparable and that there appears to be a progressive narrowing of the joint widths, which seems to be a natural phenomenon in concrete pavements, especially those constructed with intermediate contraction joints or containing continuous slabs which have cracked.

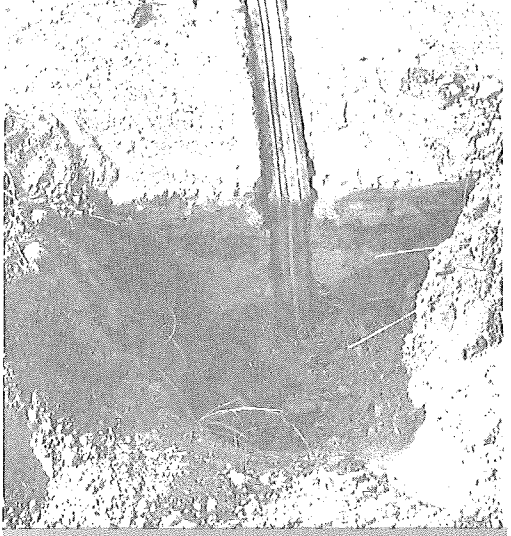
PREMOLDED FIBER FILLER VERSUS AIR-CHAMBER: Joint width measurements and visual observations indicate that the premolded fiber board has certain advantages over the air-chamber type of joint. When the fiber board is properly installed, with respect to edge of joint and surface of the slab, it provides a good foundation for the bituminous sealer. Also, since it does not extrude it will remain intact to prevent infiltration of large aggregate pieces. Furthermore, it will distribute compressive forces due to expansion uniformly over the ends of the two abutting slabs. This latter function should decrease materially the inherent tendency of concrete pavements to disintegrate at expansion joints.

Air-chamber expansion joints such as were included in the investigation require exceptionally good seals in order to perform their function satisfactorily. The air-chamber expansion joints, type DB-1 (3), located in Series 5 have developed a greater permanent closure than joints in other series constructed with premolded fiber. Since these joints offer no restraint to the adjacent slabs they have closed to approximately 50-80 per cent of their original opening. This movement has caused excessive extrusion of the bituminous seal onto the pavement surface. In two cases in Section 5G the metal inserts employed to retain the bituminous seal in place have been pushed partially out of the joint. The abnormal movement encountered in Series 5, Sections F and G may be due to two constructional factors associated with this particular test section: *first,* the two end sections F and G lie at the foot of a downgrade of 0.464 per cent; *second,* these two sections abut against a bridge structure which cannot shift laterally. In Figure 25 there are presented views of air-chamber expansion joint DB-1 (3) at the time of construction and 9 years later.

Figure 25B

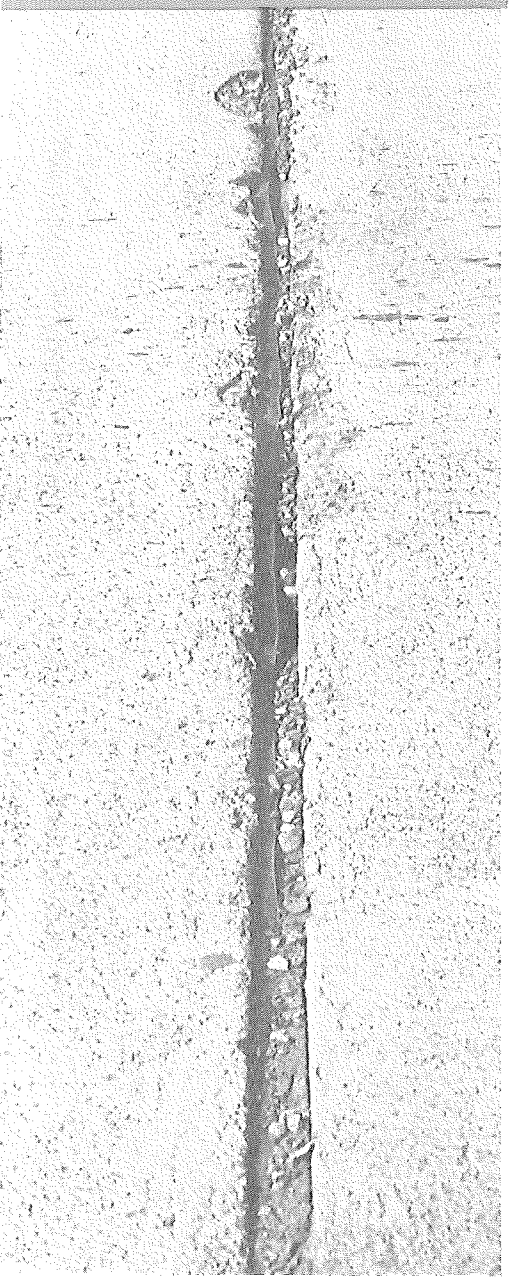
Same joint 9 years later (1949).





Air chamber expansion joint Type TB-(4), premolded rubber seal, Series 9-A, when constructed (1940).

Same joint in June 1942 when premolded rubber seal had failed. Note condition of seal material and debris in opening.



The air-chamber expansion joints used in Series 9-A, type TB-(4) have reacted very satisfactorily except for the premolded rubber seal which failed after 2 years in service. The premolded rubber seal was pushed down into the joint about ¾ of an inch and rotated 90 degrees in the joints. See Figure 26. Eventually it was necessary to remove the rubber seal, thoroughly clean the joints and reseal. In the resealing process the old premolded rubber material was placed back into the joint opening at a depth of about ¾ inch and new hot-poured bituminous-rubber joint sealing compound was poured on top to effect the seal.

LOAD TRANSFER FEATURES: The expansion joints of type A-2 which were constructed without dowels and located in Series 10A and 10B, after 4 years in service, had developed measurable faulting which has increased somewhat with age. The results of surveys made in August 1944 and July 1949 are summarized in Table VI. The values represent maximum faulting of slabs at the edge of the pavement only. All readings of 1/16 inch and under were disregarded because of the possible influence on their accuracy of normal irregularities in the pavement surface. The data in Table VI indicate very clearly the influence of mechanical load transfer devices on the prevention of faulting at expansion joints. The instrument and method used to determine the amount of faulting are shown in Figure 27.

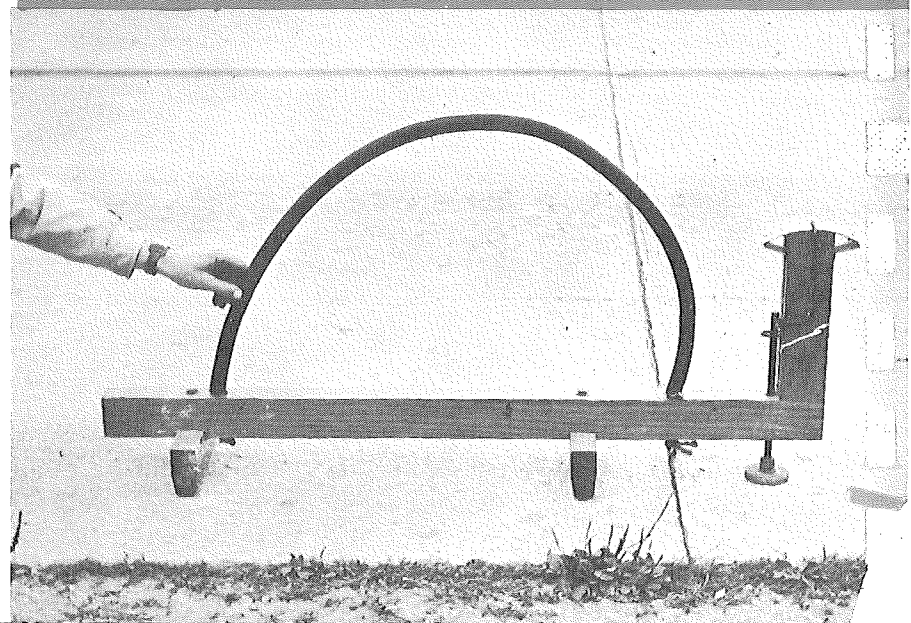
On the basis of level measurements and soil surveys, it is believed that abnormal volume changes in the subgrade are responsible to a large degree for the faulting in this section of the Test Road.

No comparative data are available in regard to the relative merits of the remaining types of expansion joint designs which were included in the load transfer and joint design study. Evidently time and traffic have not been sufficient to bring out any noticeable physical differences as yet.

Figure 26

Figure 27

Device used in measuring faulting at joints.



CONTRACTION JOINT DESIGN

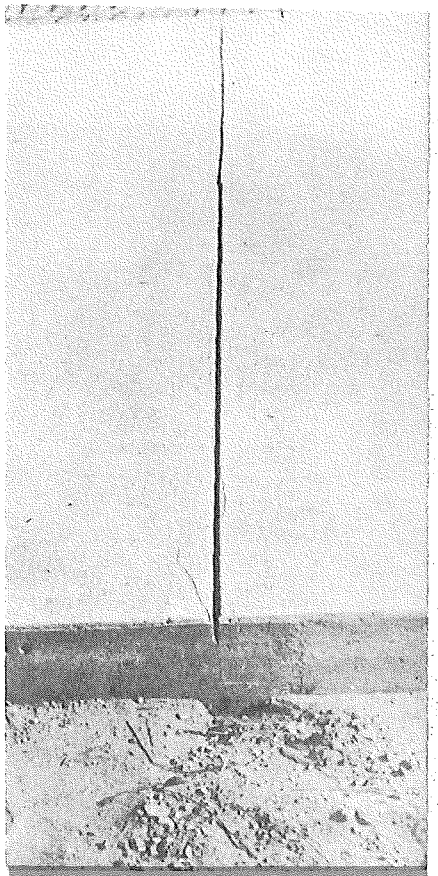
The types of contraction joint design considered in the investigation are:

- Type DB Department Standard consisting of ¾- by 15-in. dowels at 15-inch spacing with 2½- by ¼-inch. premolded fiber filler strip at top.
- Type 1B Same as above except that a ½- by 2½-in. groove is substituted for the premolded fiber filler strip.
- Type 2A Same as type DB except for the addition of a 1-in. high metal parting strip at bottom of joint.
- Type 2B Same as type 2A except the groove was substituted for the premolded fiber filler strip.
- Type 3 Metal divider plate full depth and groove at top. ¾-by 15-in. dowels at 15-in. spacing used.
- Type 4 Continuous plate dowel assembly.
- Type 5 Keylode plate dowel assembly.
- Type CB 1¼-in. by 18-in. corner bars with premolded fiber filler strip, placed 9 inches from edges.
- Type 6 Aggregate interlock only.

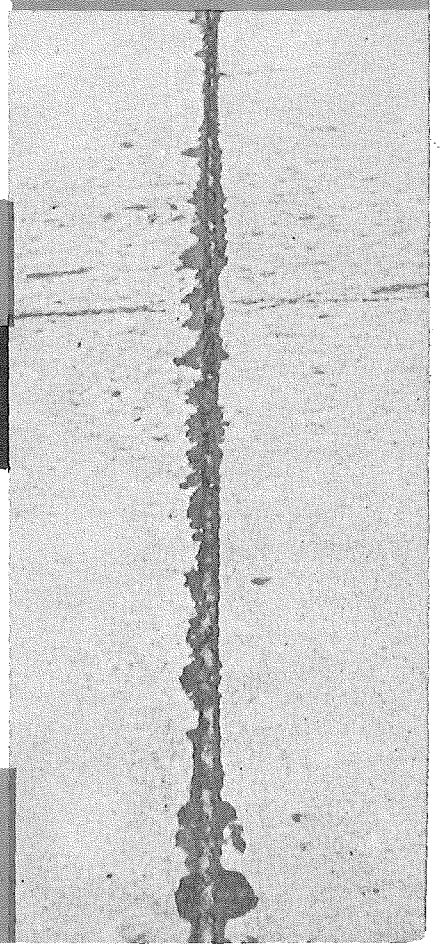
At the end of 9 years sufficient evidence has been collected on the relative behavior of the various types of contraction joint construction to warrant detailed discussion.

PREMOLDED FIBER STRIP VERSUS SEALED GROOVE: Types DB and 1B were constructed primarily to study the possibility of reducing spalling at contraction joints by substituting the groove for the premolded fiber filler strip. In that respect the following relative behavior has been observed:

In Series 5 the majority of the contraction joints were constructed with a groove which was subsequently filled with a bituminous-latex sealing compound. At the present time the joints are in excellent condition except for weathering of the seal as may be seen in Figure 28. The joint edges have remained intact and no spalling of the concrete has been noted. Some scaling has appeared at joint edges. In the case of contraction joints constructed with premolded fiber strip several kinds of failure have been encountered, the most undesirable of which is spalling of the concrete along the joint edge. Perhaps the spalling is a



Type 1B contraction joint with groove when installed in 1940, Station 64 + 90.



Condition of same joint in 1949.

Figure 28

Table VI

FAULTING OF EXPANSION JOINTS WITH AND WITHOUT LOAD TRANSFER DEVICES

TWO LANES

Series	Number of Joints Having Maximum Fault of: 1/8 inch A	1/8 inch B	3/16 inch A	3/16 inch B	1/4 inch A	1/4 inch B	Over 1/4 inch A	Over 1/4 inch B	Total Joints Faulted A	Total Joints Faulted B	Total No. of Joints in Two Lanes	Percentage of Total Faulted A	Percentage of Total Faulted B	Load Transfer
10 A-1	2	1	0	0	0	0	0	0	2	1	20	10	5	3/4- by 15 in. dowels, 15 in. spacing
10 A-2	0	0	0	0	0	0	0	0	0	0	18	0	0	3/4- by 15 in. dowels, 15 in. spacing
10 B-1	5	6	5	4	0	3	0	0	10	13	18	56	72	None
10 B-2	5	5	4	1	5	5	0	2	14	13	18	78	72	None

A — Survey August 1944

B — Survey July 1949

direct outcome of installation practice since it is usually associated with tipping of the fiber strip during installation or placing the strip too far below the surface of the pavement. Typical examples of this kind of spalling may be noted in Figure 29. In many instances it was noted that at 9 years the filler strip has become rotten and displaced by traffic. This condition is illustrated also in Figure 29. Another common fault of the premolded fiber strip is that it does not provide adequate seal, especially during the winter months when the joints are at their maximum opening. In this condition the filler strip is loose in the joint thus permitting the infiltration of water and inert material. Typical examples of this condition are presented in Figure 30. The seriousness of this condition is naturally affected by joint spacing, being greater in the sections wih 60-foot joint spacing than in the sections with 10-foot spacing.

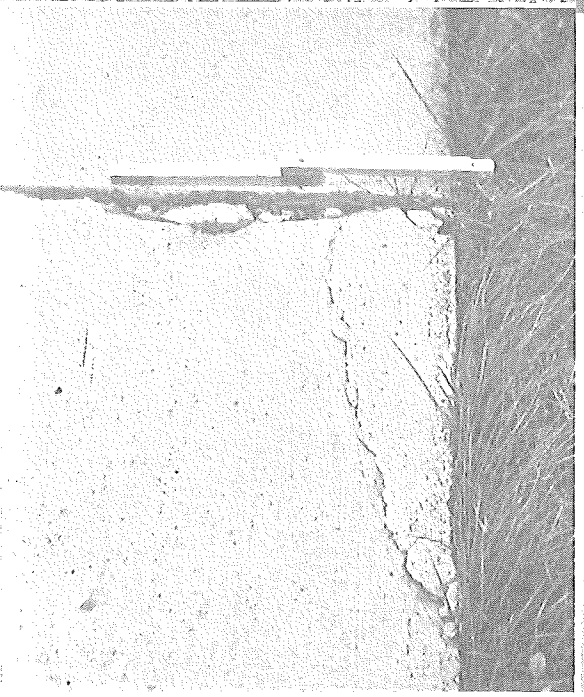
LOAD TRANSFER FEATURES

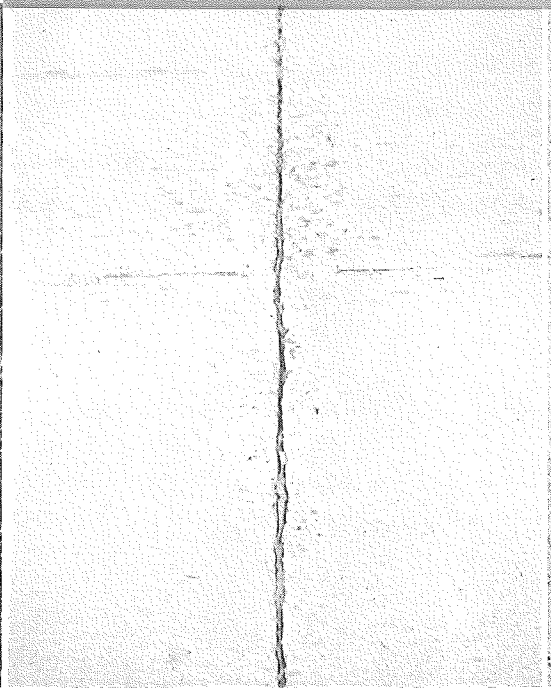
Several types of load transfer features in contraction joints were considered; standard ¾- by 15-in. dowel bars at 15-inch spacing, aggregate interlock, special corner bars and the continuous plate dowel. These features will be discussed in the order mentioned.

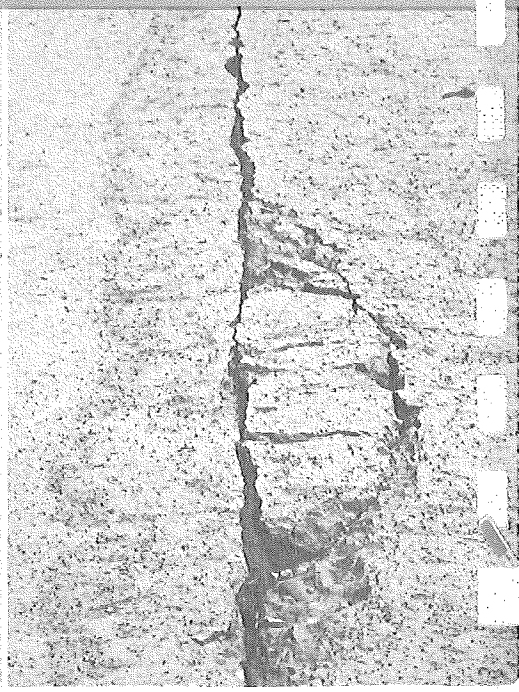
STANDARD ¾-INCH DOWEL: The standard ¾-inch dowel assembly was used throughout the Design Project with the exception of those series in which special load transfer features were incorporated for comparative study. The performance of these units will be discussed later on in conjunction with other factors associated with joint design.

Figure 29

Typical failures common to contraction joints constructed with premolded fiber strip.

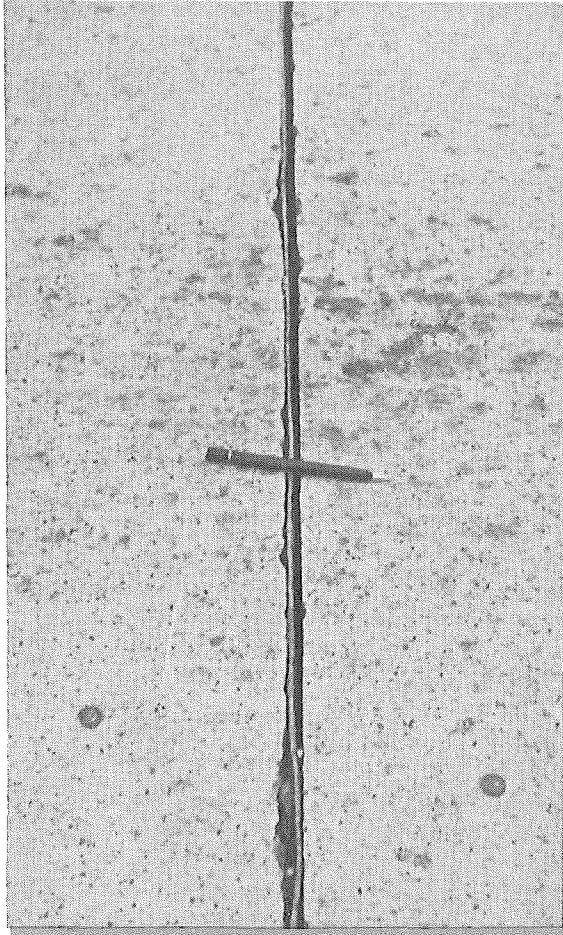






DOWEL BARS VERSUS AGGREGATE INTERLOCK: In this study two major factors were considered: *first,* the omission of dowels in contraction joints with normal expansion joint spacings of 120 feet which would naturally permit the intermediate contraction joints to open freely; and *second,* similar construction with the spacing of expansion joints at great distances which would effect considerable restraint on individual slab movement and thus develop a better joint condition for aggregate interlock to perform in the manner intended.

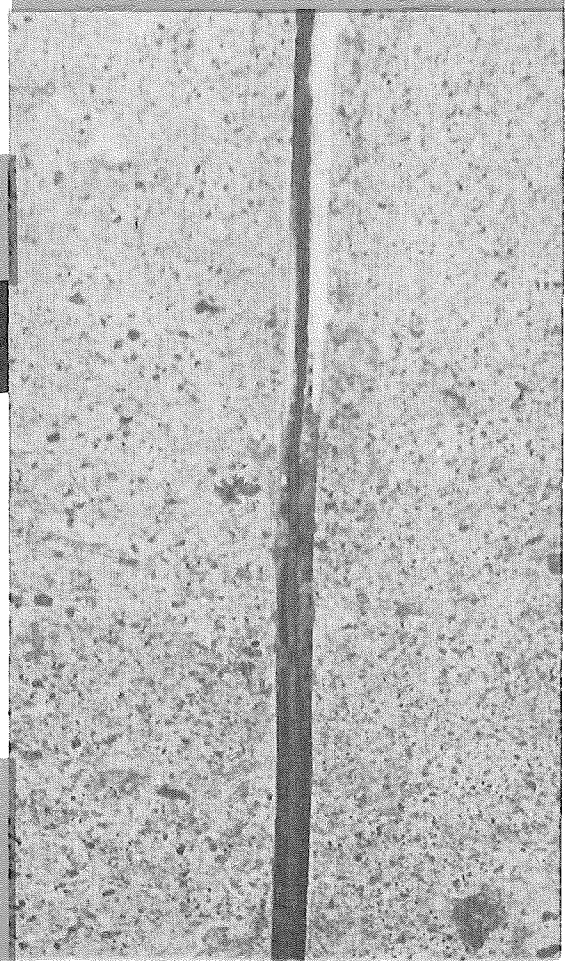
In Series 10A and 10B contraction joints were constructed with and without dowel bars at joint spacings of 15 and 20 feet, the expansoin joint spacing remaining constant at 120 feet in all sections of the series. Results of surveys conducted in the summers of 1944 and 1949 are presented in Table VII. The data in Table VII indicate that the number of faulted joints in the undoweled sections of Series 10 is considerably greater than that in the doweled sections. In all cases the degree of faulting represented indicates the amount that the corner of the approach slab was below that of the passing slab. Moreover, there is evidence that differential movement of the slabs has started at the longitudinal joints due to the absence of dowels across the transverse joint. Furthermore, the fact that a considerable number of joints with dowel bars show faulting would indicate the inadequacy of the ¾-in. dowel at such a spacing of 15 inches. The study clearly indicates the beneficial effect of dowels in contraction joint construction, especially when expansion joints are provided at distances of 120 feet as in this case.



Typical condition of contraction joints in winter with bituminous premolded fiber strip, contraction joint spacing 60 ft. Winter 1944-1945.

Table VII

Figure 30



FAULTING OF CONTRACTION JOINTS WITH AND WITHOUT LOAD TRANSFER DEVICES

TWO LANES

Series	Number of Joints Having Maximum Fault of: 1/8 inch		3/16 inch		1/4 inch		Over 1/4 inch		Total Joints Faulted		Total No. of Joints in Two Lanes	Percentage of Total Faulted		Load Transfer
	A	B	A	B	A	B	A	B	A	B		A	B	
10 A-1	7	9	0	1	1	1	0	2	8	13	90	9	15	3/4- by 15 in. dowels,
10 A-2	4	6	1	3	0	0	0	1	5	10	126	4	8	15 in. spacing
10 B-1	23	26	7	11	4	9	0	0	34	46	90	38	51	No dowels,
10 B-2	22	37	5	4	6	12	0	1	33	54	126	26	43	agg. interlock
3 D	0	5	0	0	0	0	0	0	0	5	176	0	3	3/4- by 15 in. dowels,
4 D	1	5	0	0	0	0	0	1	1	6	178	1	3	15 in. spacing
3 E	15	32	0	7	1	1	0	0	16	40	178	9	22	No dowels,
4 E	3	9	0	0	0	0	0	0	3	9	358	1	3	agg. interlock

A — Survey August 1944

B — Survey July 1949

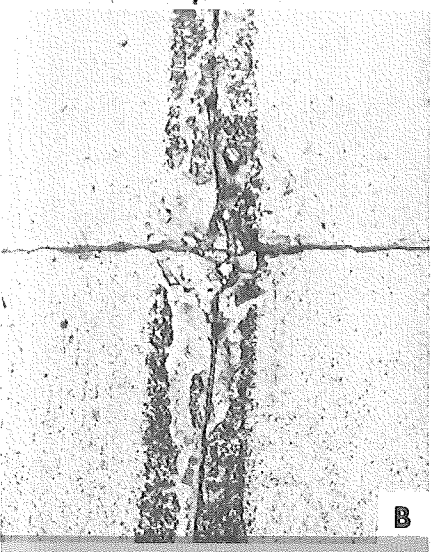
In Sections 3E and 4E dowel bars were omitted at the contraction joints for the purpose of studying the effect of slab movement restraint on the performance of aggregate interlock. The expansion joint spacing in both cases is 1800 feet and contraction joint spacings are 20 feet and 10 feet, respectively.

Sections 3D and 4D, with the same contraction joint spacing as 3E and 4E, respectively, but containing dowel bars, were chosen for comparative study. In this case the expansion joint spacing is 900 feet instead of 1800 feet. Results of surveys made in 1944 and 1949 are also shown in Table VII.

The data show in all cases a gradual increase in faulting with time. Faulting of the 20-foot slabs in Series 10 with no slab restraint is considerably higher than that in Series 3E with some slab restraint. However, even in the latter case, it is clearly indicated that aggregate interlock is not entirely effective in preventing faulting. Further, the data for Series 4D and 4E indicate that aggregate interlock is beginning to lose its effectiveness in the more restrained sections also.

The absence of dowels has created a weakness in the pavement structure at the junction of longitudinal and transverse joints which eventually may give rise to a serious maintenance problem. This weakness is manifested by noticeable differential movement of the slab corners at the intersection of the transverse joints and the longitudinal joint which has resulted in spalling of the concrete at the joint intersection and is slowly progressing along the longitudinal joint. However, since 1945 spalling at the same location in doweled joints has occurred to a considerable extent where premolded joint seal was used. Figure 31 presents typical contraction joint conditions with and without dowels.

CORNER BARS: In Series 6 and 8, 1¼- by 18-in. corner bars, placed 9 inches in from the slab edges were substituted for the standard ¾-in. dowel bar assemblies at all contraction joints. At the present time there is no discernible physical condition of the joints which would indicate that they are not performing in a satisfactory manner.



Typical spalling at joints of Section 3E, 20-foot spacing, no dowels.

A. Spalling at intersection of longitudinal and transverse joints.
B. Spalling along longitudinal joint.

Typical spalling occurring on sections with dowels, Series 1F, Station 839 + 60.

Corner break at Station 1000 + 70, Section 3E, no dowels.

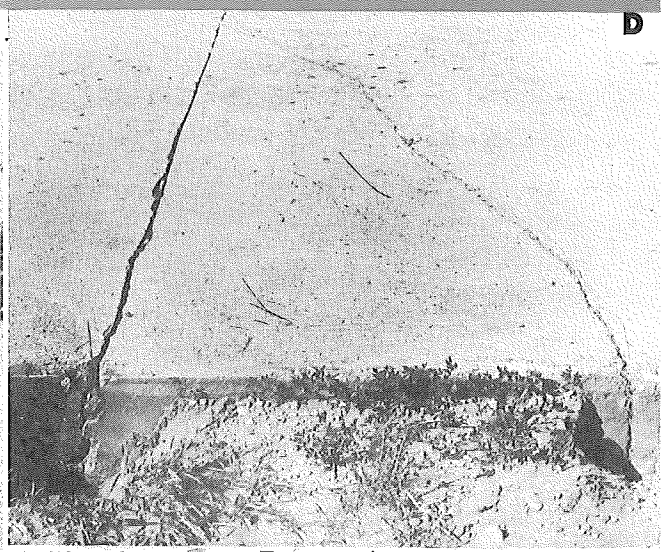


Figure 31

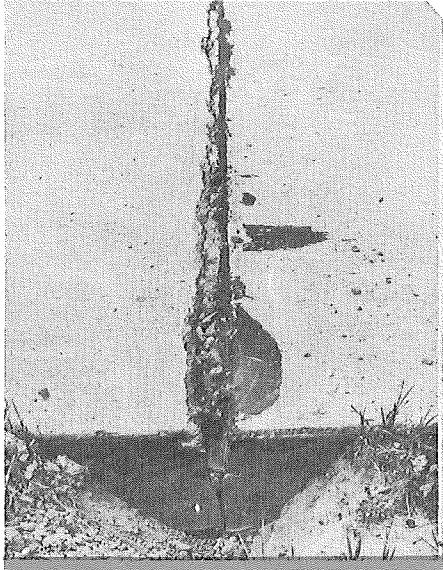
CONTRACTION JOINT PERFORMANCE WITH AND WITHOUT DOWELS

CONTINUOUS PLATE DOWEL: Two types of continuous plate dowel assemblies in common use at the time were considered for comparative study. One particular unit, designated as Type 5 (Keylode) and employing aggregate interlock in conjunction with a plate dowel was installed at three contraction joints in Section 9-TS between Stations 180 + 10 and 181 + 90. The other plate dowel unit known as Type 4 was installed at all contraction joints in Series 5, Secions F and G.

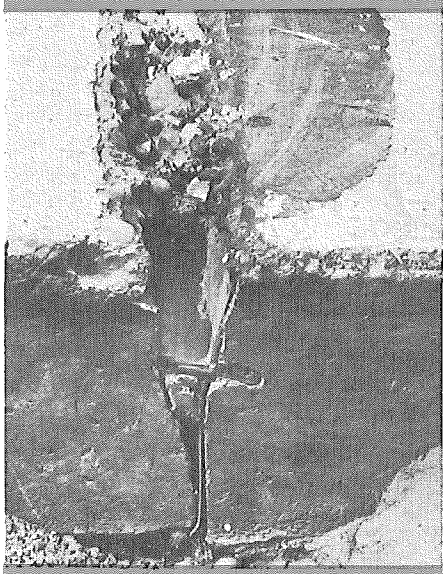
Views illustrating typical conditions of Type 5 joint at construction and 9 years later are presented in Figure 32. In Figure 32, two physical weaknesses are in evidence. In the first place the joint assembly provides no seal at top or sides against the infiltration of water or inert material. Note how the upper portion of the joint opening is full of inert particles. Second, it is apparent upon examination that the projections of concrete which extended to form the aggregate interlock are broken, thus destroying any load fransfer action from that source.

The continuous plate dowel, Type 4 located in Series 5, has an inherent design weakness which causes spalling along the joint edge. Examples of the type of spalling may be seen in Figure 33. An examination of the joint revealed that the plates invariably were frozen in place due to rust and consequently the joints were not functioning as designed. The diagrammatic sketch in Figure 34 shows the general condition of the pavement in Series 5 and especially the degree of spalling in Sections F and G caused by the plate dowel assemblies.

CRACK CONTROL METHODS: The examination of contraction joints in many concrete projects as well as those in the Michigan Test Road revealed that the spontaneous cracking of the pavement at the plane of weakness joints was very irregular and in general not vertical as desired. Also it was observed that in many cases diagonal cracking and subsequent spalling was common at the bottom of the contraction joints. A typical illustration of cracking and spalling at the contraction joint is shown in Figure 35.



Typical spalling at joint edge due to continuous plate dowel assembly (1946), Station 83 + 20.



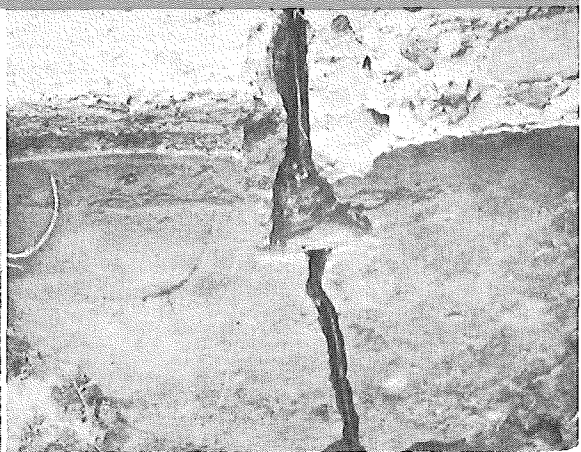
End condition of joint with continuous plate dowel assembly (1946).

Figure 33 Type 4

Figure 32

Typical condition of Type 5 contraction joint at time of construction (1940), Station 181 + 60.

Typical condition of all similar joints after 9 years, Station 181 + 60.



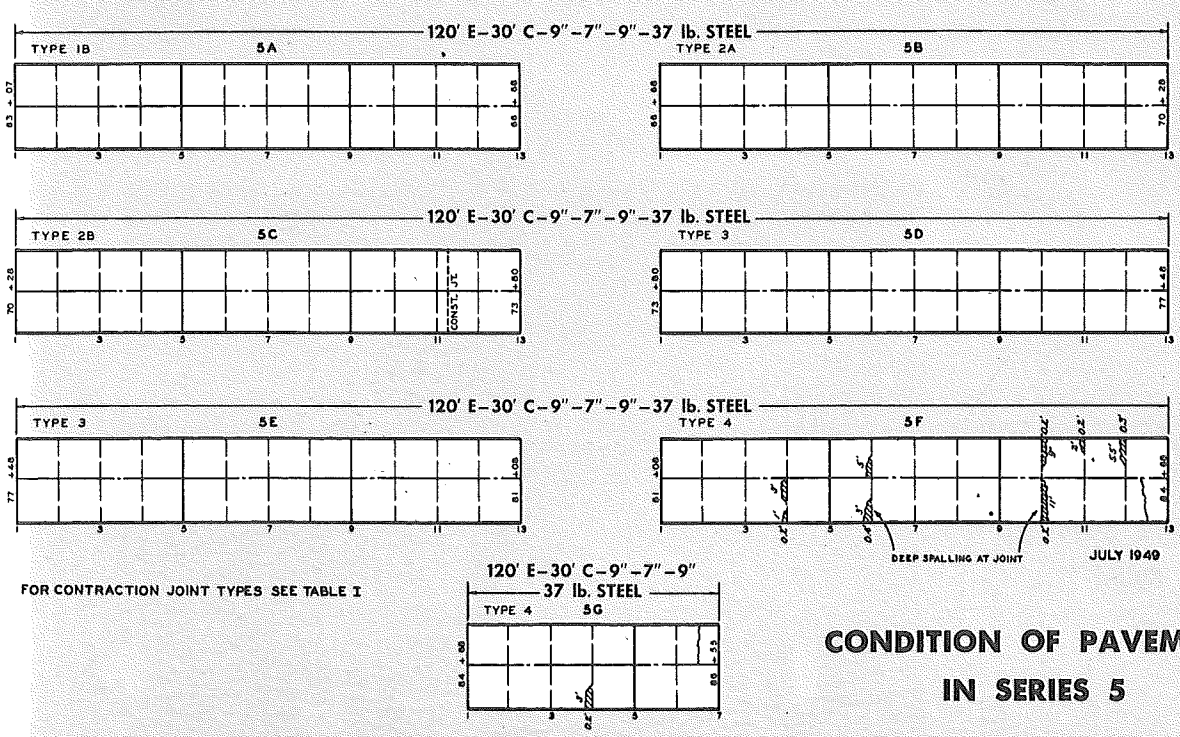
CONTRACTION JOINT WITH CONTINUOUS PLATE DOWEL, TYPE 5

With these facts in mind, two well-known devices for controlling cracking of the pavement at contraction joints were installed in Series 5 for comparative study. One method employs a one-inch metal parting strip installed in the bottom of the pavement directly under the premolded fiber filler strip or the groove, whichever is being used to form the plane of weakness. These joints are designated Types 2A and 2B. The other type of joint construction employs a vertical 22-gage continuous metal plate extending the full depth of the pavement to break the continuity of the concrete. In this case, the metal dividing plate was used with the groove and is designated as Type 3.

There is no question that the full continuous metal plate will insure positive cracking of the joint, but in order to attain proper results with the metal parting strip placed at the bottom of the joint careful workmanship must be exercised. As illustrated in Figure 35 methods must be employed that will insure the proper placing of the metal parting strip directly under the premolded filler strip or groove; otherwise undesirable cracking will result. When properly placed, vertical cracking of the pavement will take place.

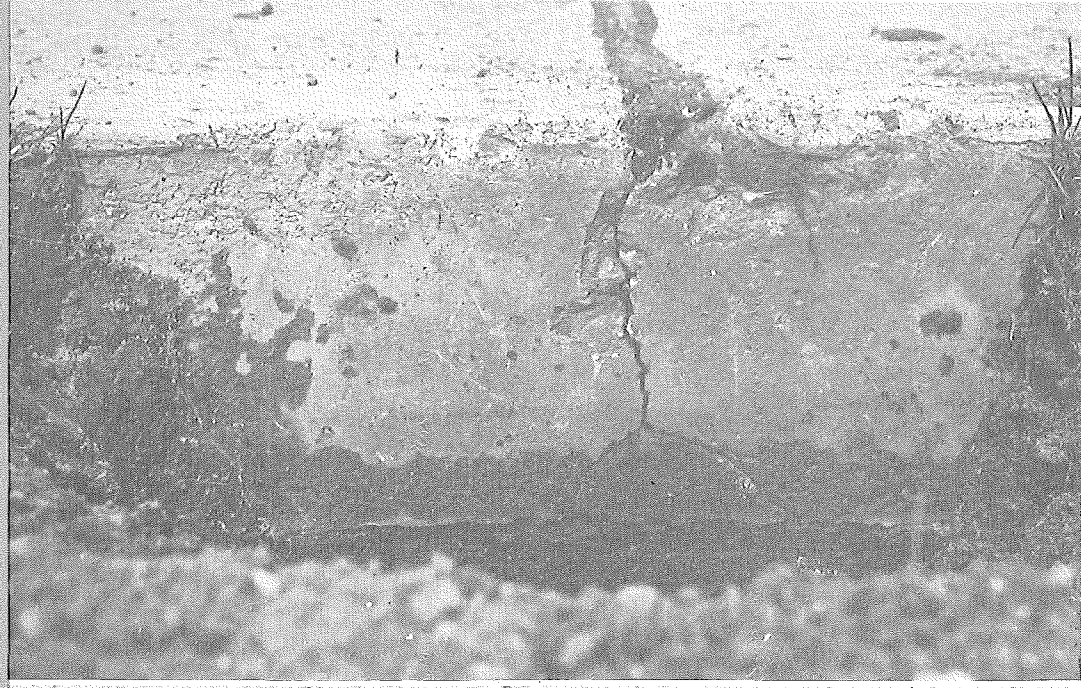
In review, the data presented in connection with the design of transverse joints disclose several significant facts. They are: *first,* from the standpoint of both construction and performance the non-extruding premolded fiber board expansion joint has many more desirable features than the air chamber type of expansion joint used on the project; *second,* due to the large seasonal movement of the contraction joints, aggregate interlock in itself is not sufficient to provide adequate load transfer to prevent faulting of slabs; *third,* the practice of providing ¾-inch dowels at 15-inch spacings is not adequate to prevent faulting of slabs either at expansion or contraction joints; *fourth,* certain types of plate dowel load transfer units are to be avoided; *fifth,* the premolded fiber strip used in forming contraction and dummy joints is a detriment to good concrete pavement construction. Unless great care is exercised both in the placing of the strip and during subsequent finishing operations, irreparable damage results to the pavement.

Figure 34

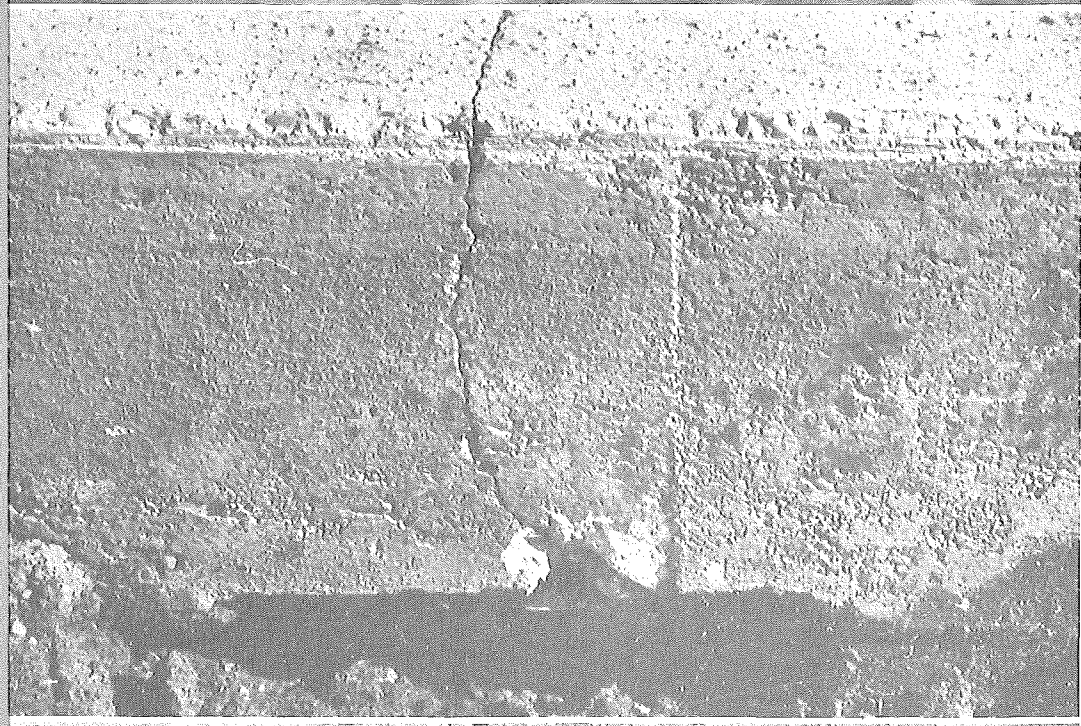


CONDITION OF PAVEMENT IN SERIES 5

Typical cracking at plane-of-weakness joints without metal parting strip at bottom.



Typical cracking when metal parting strip is not properly installed.



Typical cracking when metal parting strip is properly installed.

EFFECT OF PARTING STRIP ON JOINT CRACKING

Figure 35

PAVEMENT CROSS SECTION

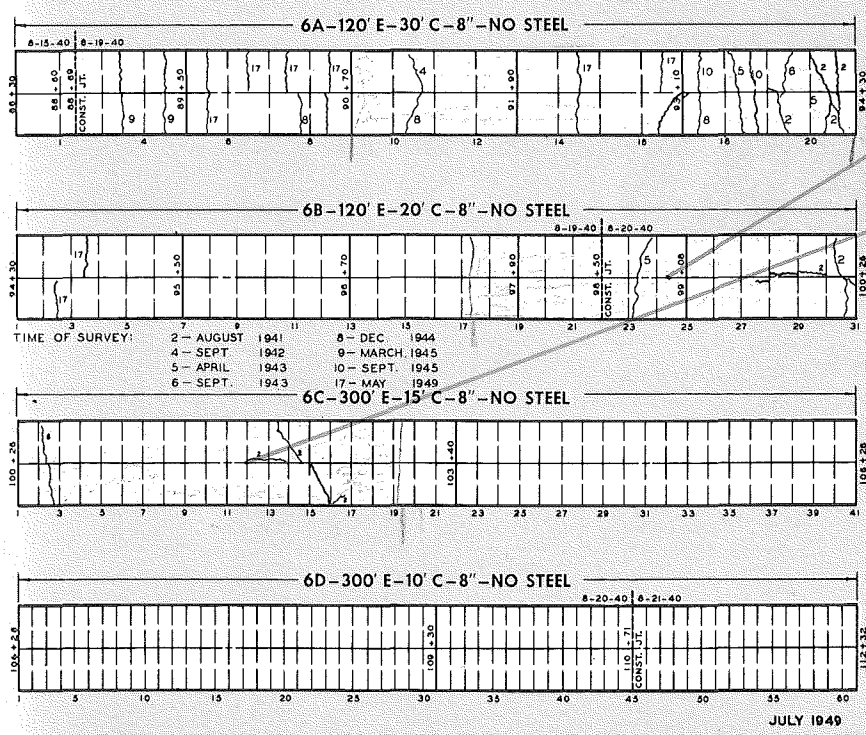
Four different types of pavement cross sections were included in the Design Project for the purpose of studying such factors as load capacity of subgrade versus slab thickness and the balanced or thickened edge cross section versus equivalent uniform thickness. The cross sections set up for study include the 9- 7- 9-in., and its approximate equivalent 8-inch uniform; the 8- 6- 8-in., and its approximate equivalent 7-inch uniform. The portions of the Design Project devoted to this study are Section 3A and Series 6, 7, and 8.

In general nothing of note has developed so far in any of the series involved from which comparative or conclusive data can be established. The joints and slabs in all sections have, after nine years, shown no marked difference in their relative behavior due to normal service conditions. The study does, however, emphasize how very important it is to exercise rigid examination, inspection, and control over the preparation of subbase and subgrades for concrete pavement construction.

PHYSICAL CONDITION OF SECTIONS

In the winter following the construction of the Test Road some cracking developed in part of Series 6 from Station 90 + 70 to 94 + 30 and 97 + 60 to 103 + 00. During the first four years a considerable number of longitudinal and transverse cracks continued to develop in this area as illustrated in Figure 36.

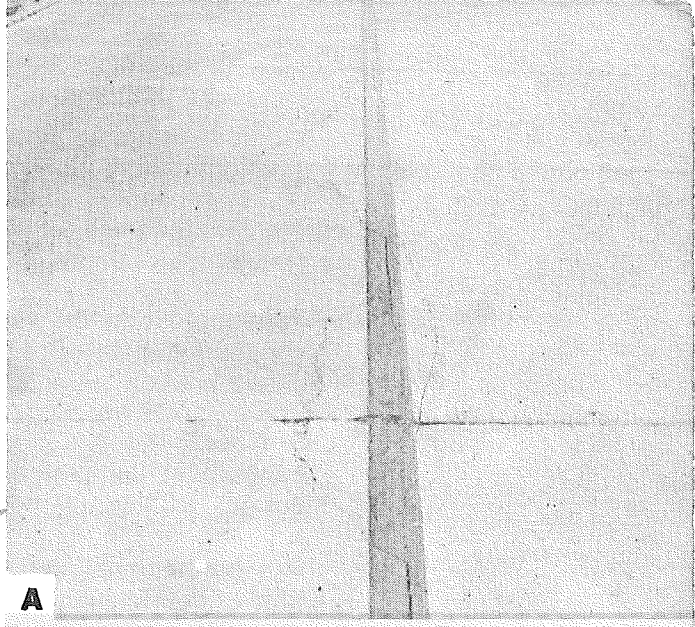
Figure 36



CONDITION OF PAVEMENT IN SERIES 6

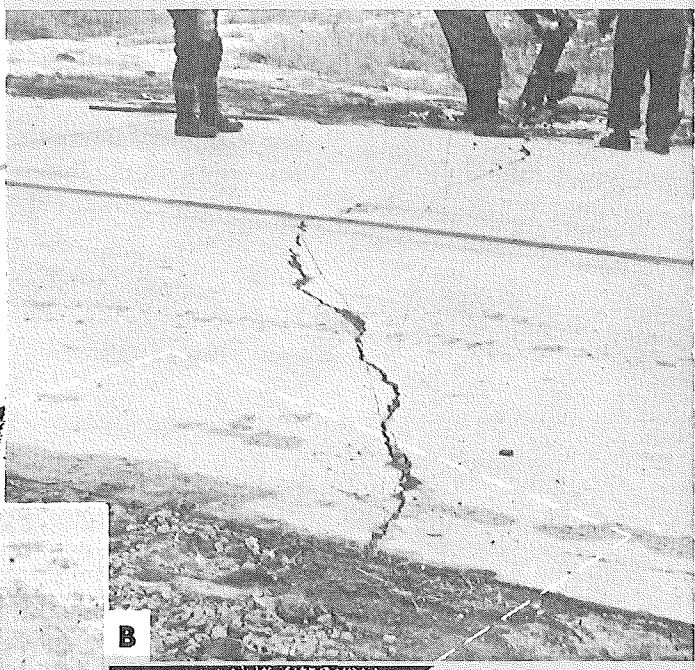
The crack pattern throughout the cracked area is typically that caused by heaving of the pavement due to volume changes in the subgrade. The pavement in Series 6 was constructed on a 12-inch sand subbase overlying Emmet loamy sand and Isabella loam. A soils survey in the spring of 1944 revealed several factors contributing to the abnormal cracking. In the first place the sandy clay subgrade material was badly rutted, protruding into the subbase material practically the full depth in some places. Such a condition would naturally prevent the normal lateral drainage of the subbase material; consequently, water pockets formed directly beneath the slab, which resulted in the longitudinal cracking of the slab as shown in Figure 37.

In addition to the faulty subbase condition it was learned that the subgrade material in the cracked sections contained pockets and laminations of peat, clay and silt, all of which undergo considerable volume change during freezing and thawing of the subgrade. Ice lenses were readily discernible in soil samples from test



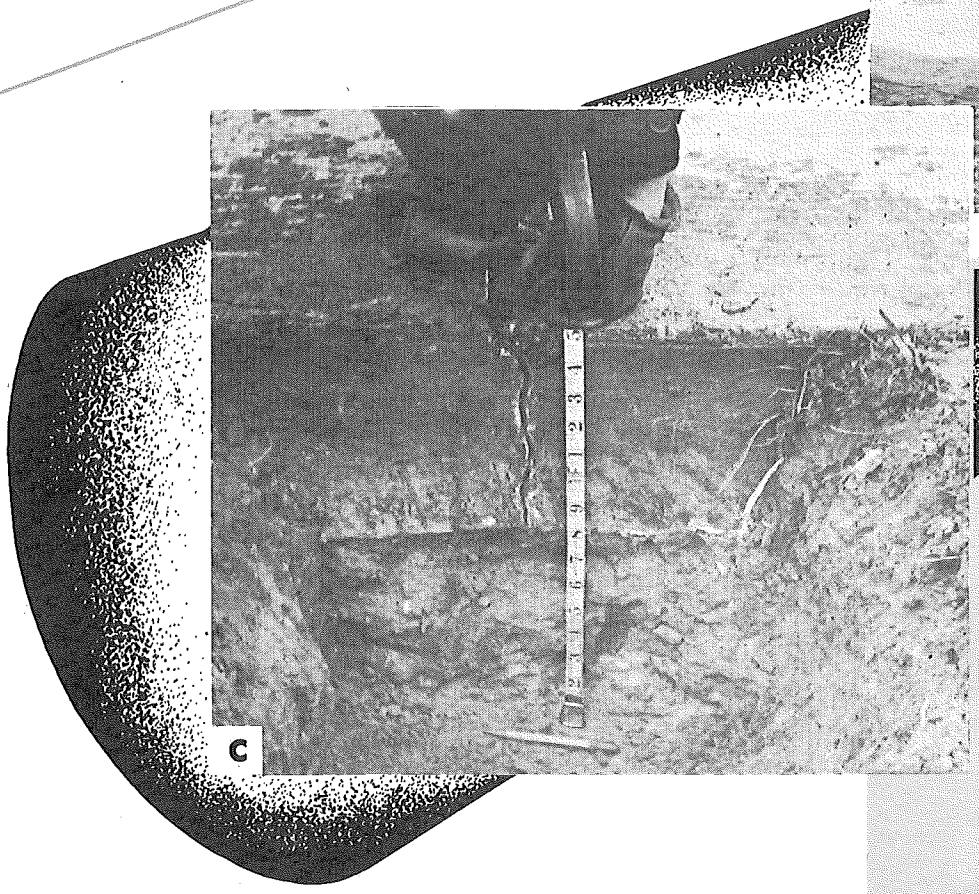
A

Longitudinal cracking caused by frost heave, Series 6, Station 99 + 00.



B

Transverse crack caused by frost heave, Series 6, Station 102 + 00.



C

Figure 37

End view of crack at Station 102 + 00. Note depth of sand sub-base (0.8 foot) over clay sub-grade.

holes. Such subgrade and subbase soil conditions are no doubt responsible for the preponderance of transverse and diagonal cracks within these areas.

Transverse cracks are beginning to appear, however, in other areas of Series 6. In Figure 36 it will be noted that seven of the 30-foot slabs in Section 6A have transverse cracks which are not believed associated with foundation conditions.

In Series 7, with 8- 6- 8-in. cross section, the first transverse crack was noted in December 1944, and since that time several more have developed as may be seen in Figure 38.

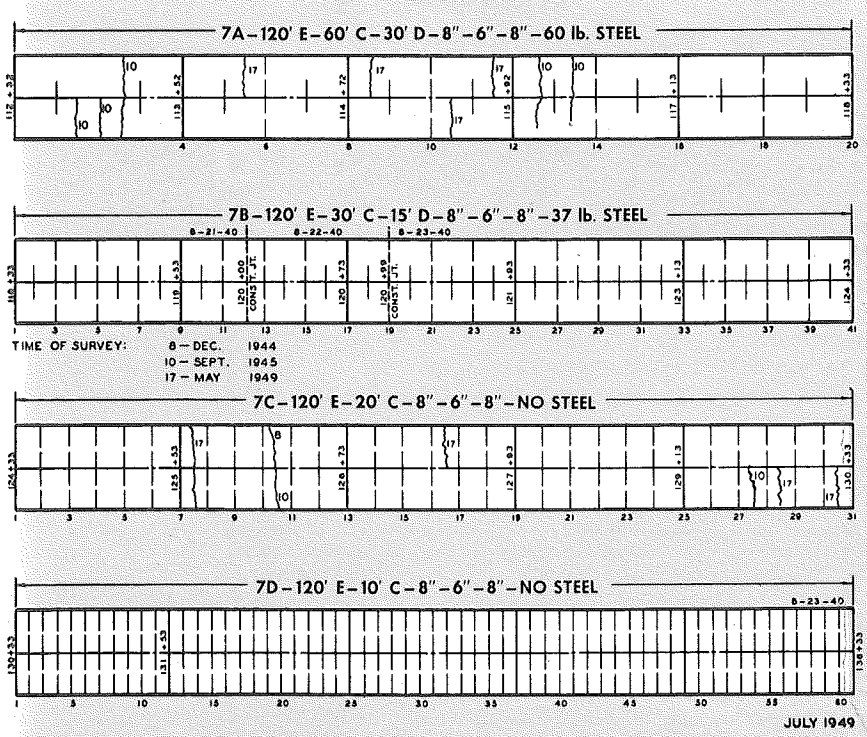
Only two transverse cracks have developed in Series 8 with 7-inch slab thickness. The first was observed in April 1943 and the last in December 1944. See Figure 39.

EXPANSION AND CONTRACTION JOINT MOVEMENT

In Series 6, 7, and 8 expansion and contraction joint spacing were considered in conjunction with cross section design. The expansion joints are spaced at 120 feet and 300 feet with contraction joints at 30, 20, 15, and 10 feet. Also the sections were not reinforced with steel mesh.

The joint width measurements at the present time, as previously mentioned under "Concrete Pavement Design Studies," indicate no significant relationship between

Figure 38



CONDITION OF PAVEMENT IN SERIES 7

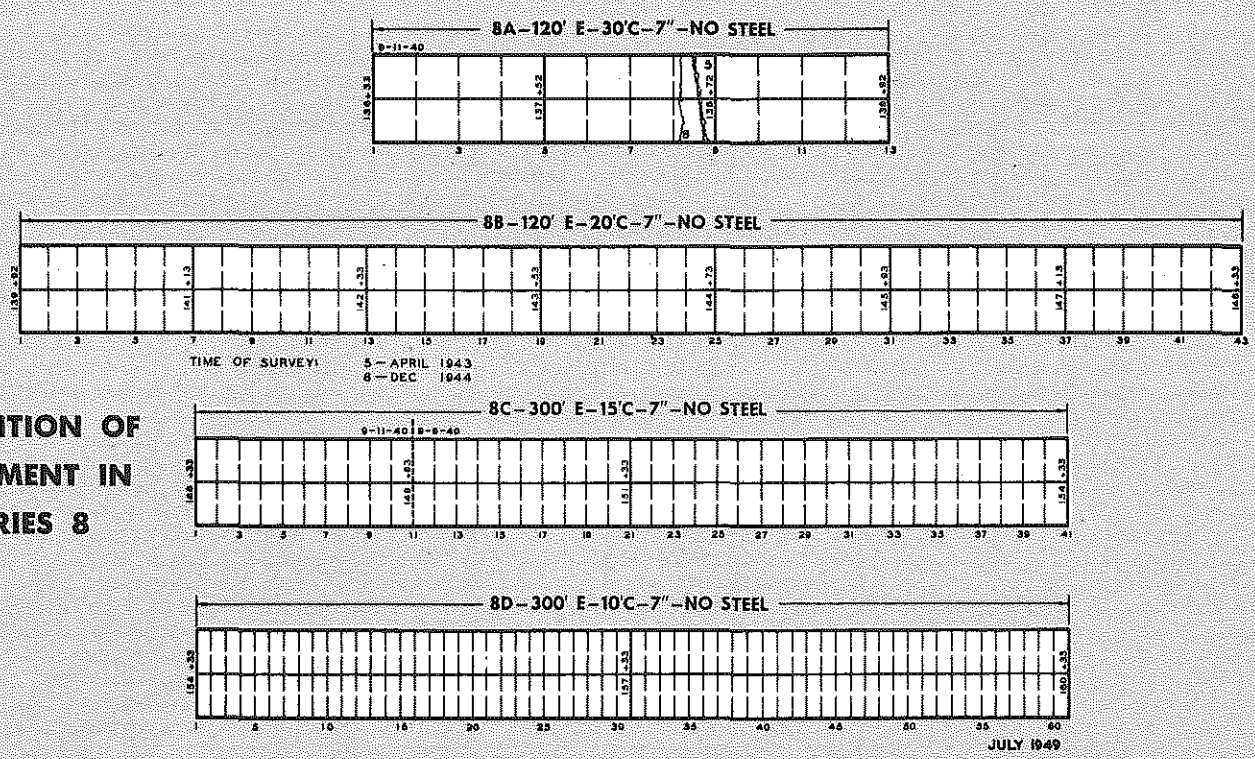
joint movement and thickness or shape of pavement cross section. The joint movements in Series 6, 7, and 8 duplicate very closely the magnitude, annual amplitude and progressive displacement of the joints located in various series containing similar expansion and contraction joint spacing.

STEEL REINFORCEMENT

Consideration was given to the problem of designing pavements with and without steel reinforcement. To this end different sections of the Design Project were constructed with 60, 37, and 0 pounds of steel reinforcement per 100 square feet of pavement. The problem of reinforcement was also considered in connection with contraction and dummy joint construction as well as in the construction of continuous slabs of varying lengths without intermediate contraction or dummy joints. Included in the study are such factors as amount of reinforcement and the influence of reinforcement on subsequent structural integrity of the pavement.

With the exception of the few areas previously discussed which are showing abnormal stress due to faulty subgrade conditions, the pavement throughout the balance of the design project is apparently in excellent structural condition. Therefore, no comparative data are available for judging the merits of different amounts of reinforcement in keeping the various slabs intact.

Figure 39



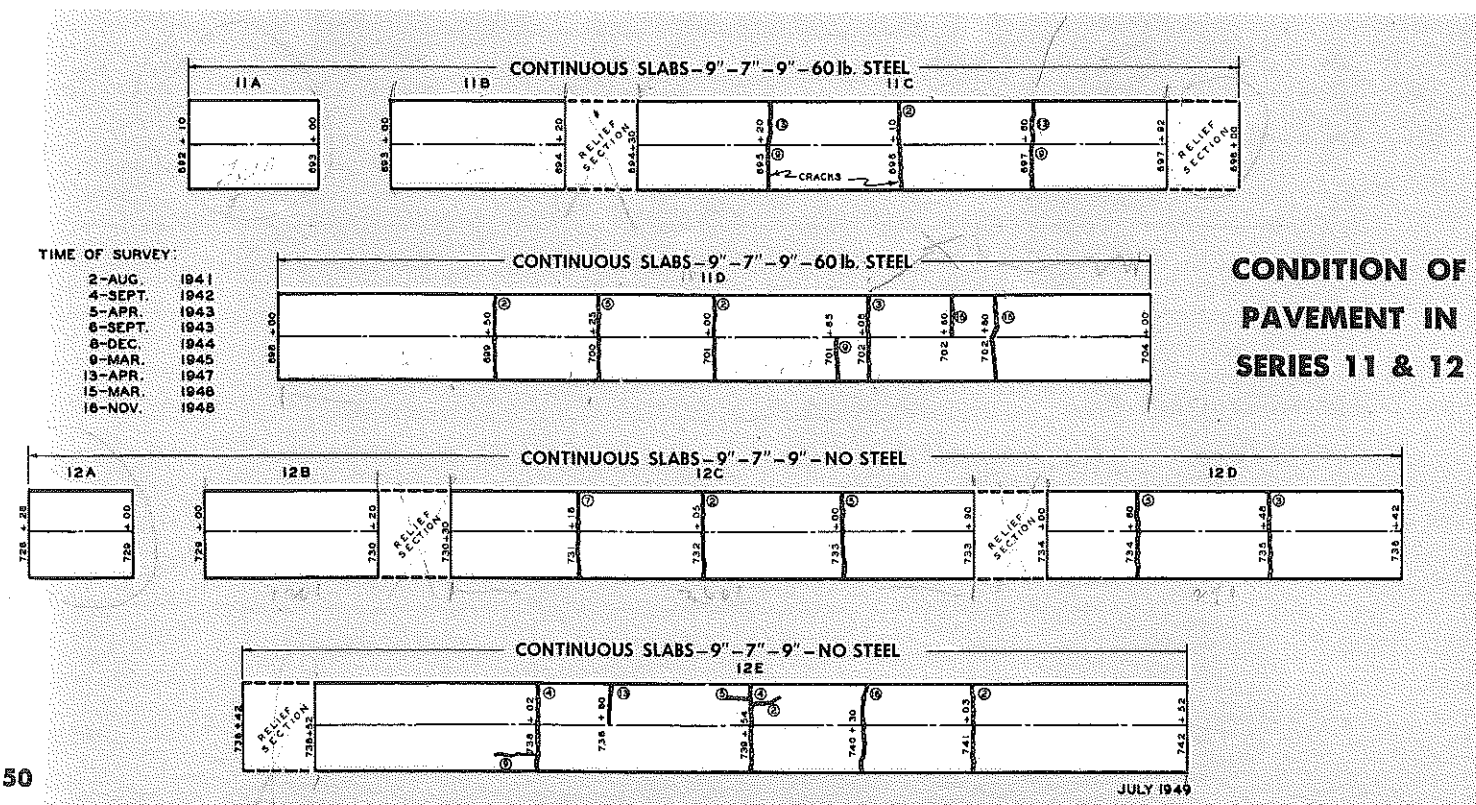
CONDITION OF PAVEMENT IN SERIES 8

CONTINUOUS SLABS WITH AND WITHOUT REINFORCEMENT

Two sections, designated Series 11 and 12 of the Design Project, were constructed in conjunction with the Durability Project of the Test Road. See Figure 3 and Table I. These two sections of pavement included continuous slabs of different lengths with and without reinforcement. Series 11 and 12 were established in order to obtain more comprehensive data relative to concrete pavement design, especially in relation to the behavior of continuous slabs versus slabs with intermediate contraction and plane of weakness joints and for slabs constructed with and without reinforcement. Special attention has also been given to changes in slab length, progressive cracking of the slabs, and the influence of steel on degree and character of cracking. Each series contains continuous slabs of lengths 90, 120, 360, and 600 feet. Steel reinforcement at 60 pounds per 100 square feet was placed in Series 11. Series 12 was not reinforced. It is to be noted that these sections were not built by the contractor of the Design Project and the materials, such as cement and aggregates involved in the construction of the concrete slabs, were obtained from entirely different sources. It is believed that these factors will have very little effect upon the final results derived from the study.

PHYSICAL CONDITION OF THE SLABS: The physical condition of the slabs in both series with respect to cracking is shown in Figure 40. The crack pattern of both series is very similar. For example, the 120-foot sections have not cracked and the 360-foot sections of the two series are cracking in a similar manner, but the full transverse cracks have formed sooner in the unreinforced section. In the 600-foot sections the reinforced section has developed six full transverse cracks whereas

Figure 40



the unreinforced section has only four full cracks. Perhaps the most remarkable thing noted from this study is the continued resistance of the long slabs, both reinforced and unreinforced, to further breakup.

The general character of the cracks may be seen in Figure 41. With few exceptions the cracks formed first at the monument boxes which were set in the slabs to measure slab movement. The maximum width of crack opening in the slabs varied from ¼ inch to ½ inch or more. From visual examination and crack width measurements, it is apparent that the steel mesh in Series 11 has broken at the cracks.

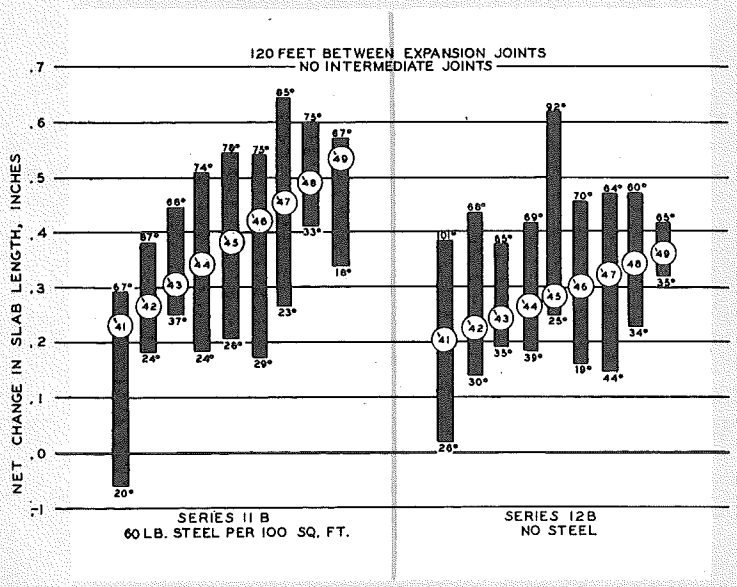
SLAB MOVEMENT: Reference monuments were installed at the ends, center, and quarter points of the slabs in Series 11 and 12 to observe their movements over a period of years. Unfortunately, the subsequent cracking of the long sections has reduced them to a series of short slabs which has restricted somewhat the original purpose of the study. Seasonal readings were continued, however, and the slab movement at the various monuments has been presented graphically in Figures 42 and 43. In general, the data further show an influence of restraint on slab movement similar to that observed on the 2700-foot sections in Series 1, 2, 3, and 4.

Since the 120-foot slabs of Sections 11B and 12B have not broken, it is possible to observe the net change in their length. The net seasonal and progressive changes for the 9-year period are shown in Figure 44. It may be observed from the plotted data in Figure 44 that both slabs have acquired a residual increase in length of approximately 0.3 to 0.4 of an inch. The greatest net change in slab length apparently took place during the first year, whereas during the succeeding years the increase in length has been gradual and much smaller in amount. The values plotted represent field measurements at the existing slab temperatures indicated for winter and summer seasons.

Figures 42 & 43

ANNUAL AND PROGRESSIVE CHANGES IN SLAB LENGTH

Figure 44

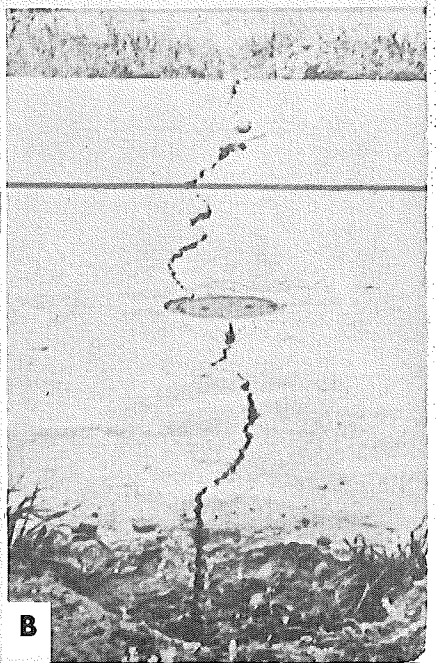


Typical condition of transverse cracks. Note exceptional width of crack opening, Section 12E, Station 739 + 54.

Figure 41A

Typical cracking at monuments for determining slab movement, Section 11C, Station 696 + 10.

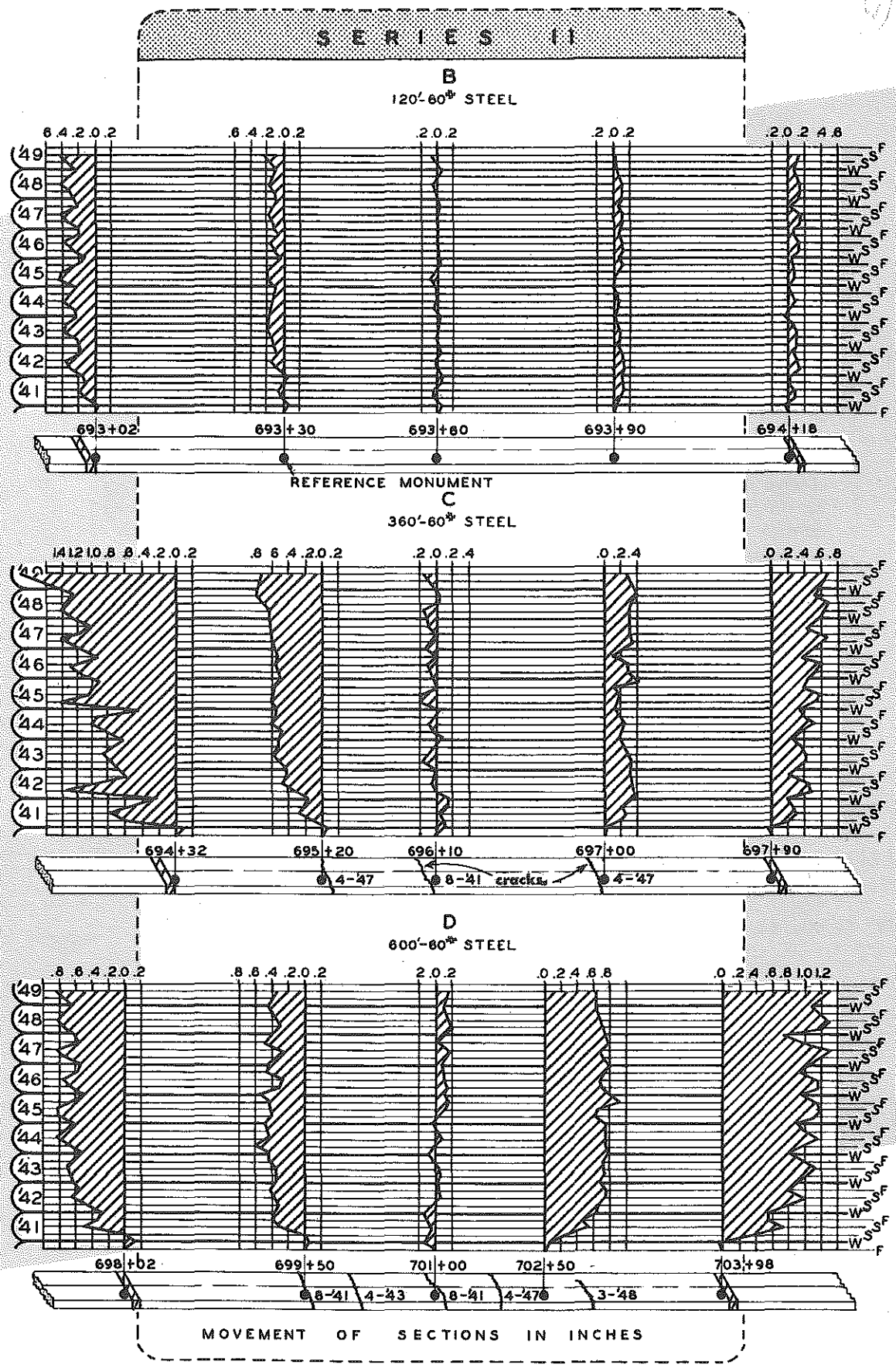
Figure 41B



A

B

Figure 42


SERIES 11
B
120'-60# STEEL
REFERENCE MONUMENT
C
360'-60# STEEL
cracks
D
600'-60# STEEL
MOVEMENT OF SECTIONS IN INCHES

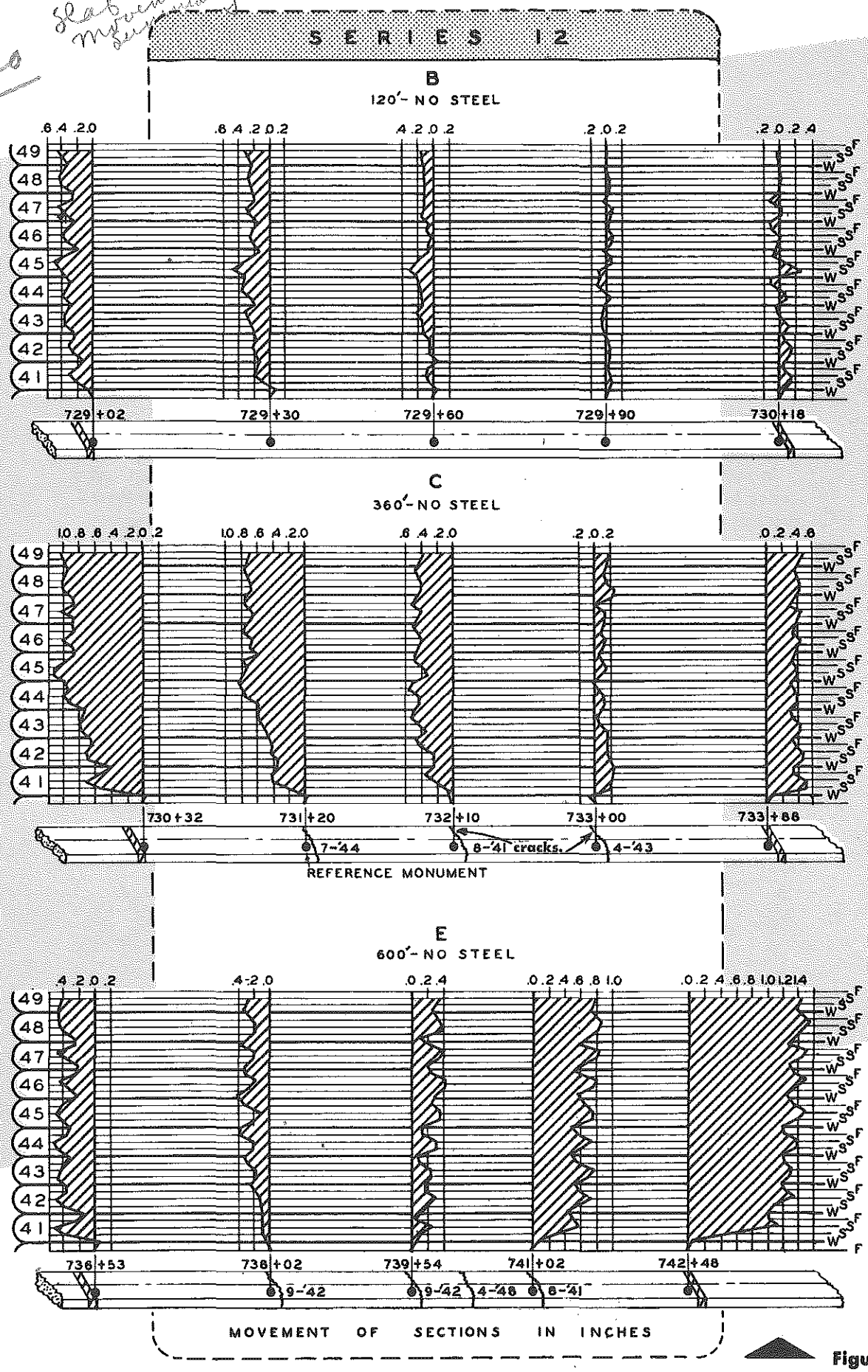


Figure 43

SEASONAL CHANGES IN SECTION LENGTH

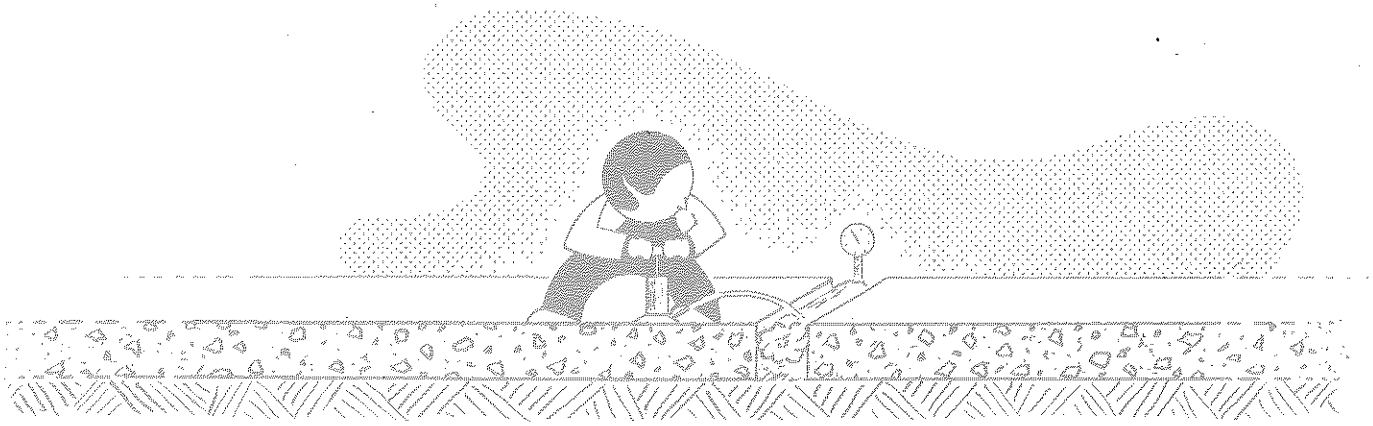
INCIDENTAL STUDIES

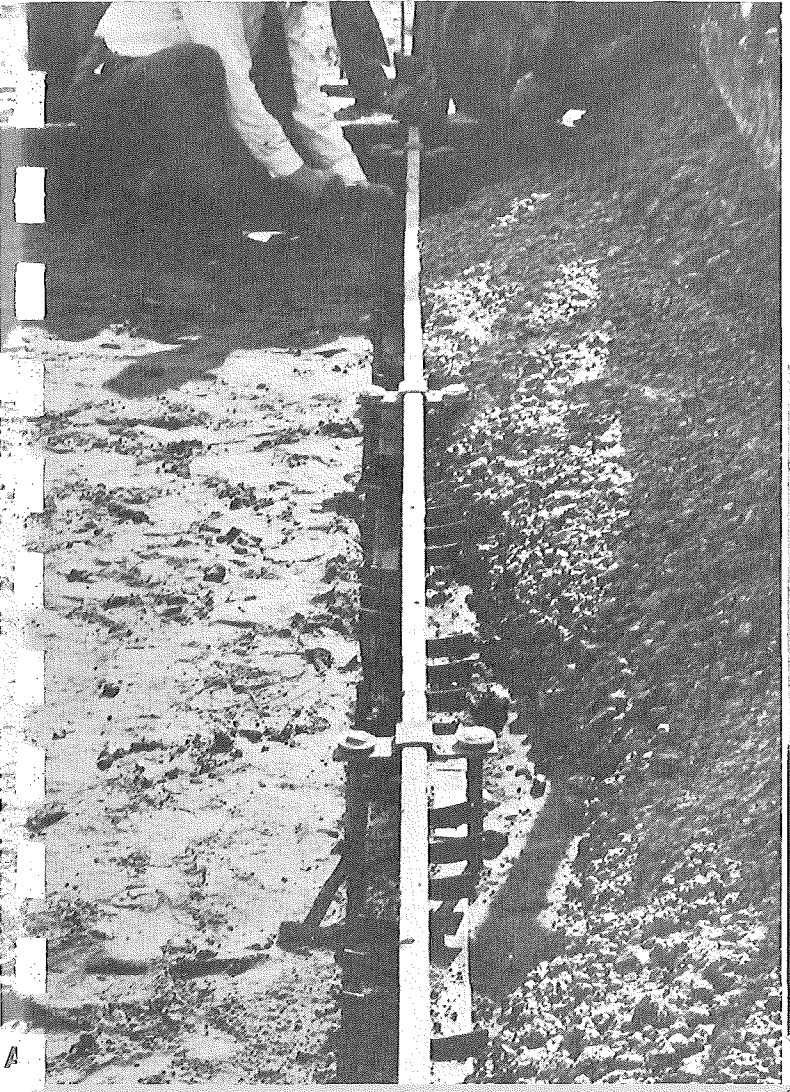
In addition to the major investigations embodied in the Michigan Test Road, several incidental studies were introduced into the program which were of particular interest to the Michigan State Highway Department. These studies pertained primarily to various construction methods, such as stress curing of concrete, mechanical spreading of concrete, mechanical form tamping, and joint sealing compounds. Several phases of pavement behavior were studied also, including roughness, moisture condition of subbase and pavement, vertical displacement of the slabs, strain in pavement, and physical condition of the pavement.

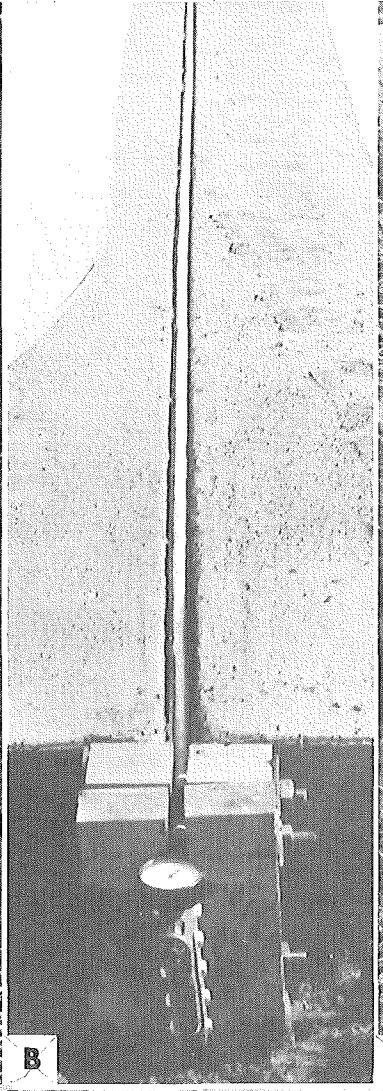
STRESS CURING OF CONCRETE

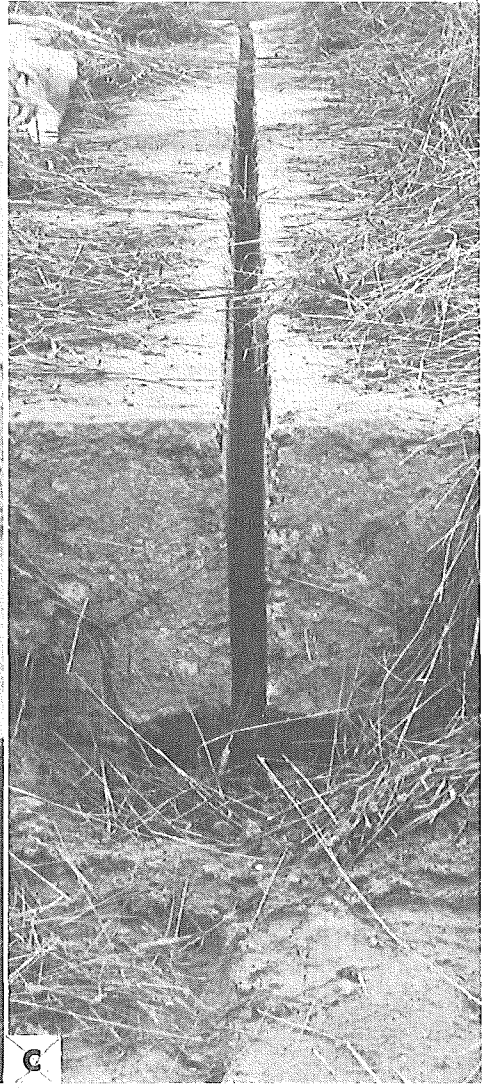
In Series 9A, eighteen hundred feet of concrete pavement was placed by the stress curing method, which eliminates reinforcement and transverse joints other than those provided for expansion. The slabs were laid in 100-foot lengths. The concrete was subjected to controlled compressive forces during the 7-day curing period, or until such time as the beams for modulus of rupture tests reached the 7-day specification strength of 550 pounds per square inch. The application of pressure was accomplished by using canvas covered rubber hose pressure cells inserted in the expansion joint openings. The pressures were increased at a rate controlled by determinations of strength increase in test specimens up to a maximum of 200 pounds per square inch. See Figure 45.

PHYSICAL CONDITION OF SLABS: At the end of 9 years, 10 of the 18 slabs are in apparently perfect condition. The remaining 8 slabs have cracked as shown in Figure 46. The first crack in the entire test section occurred in the slab between Joints 6 and 7 prior to the survey conducted in August, 1941. The progressive development of cracks is also illustrated in Figure 46 by the numbers appearing at each crack.





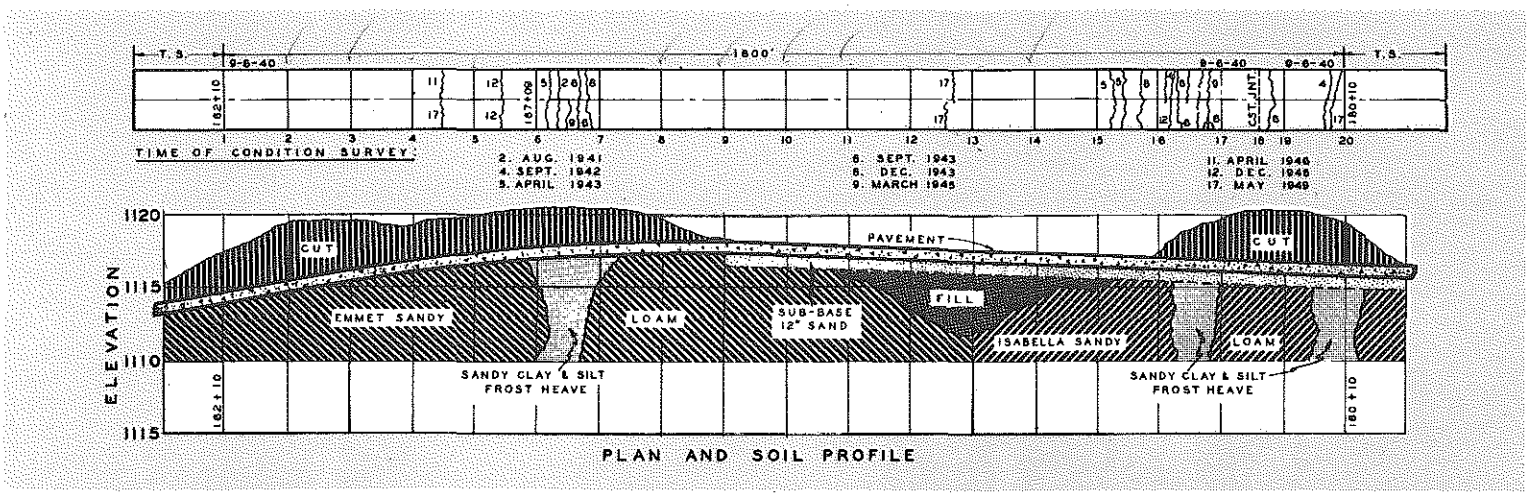


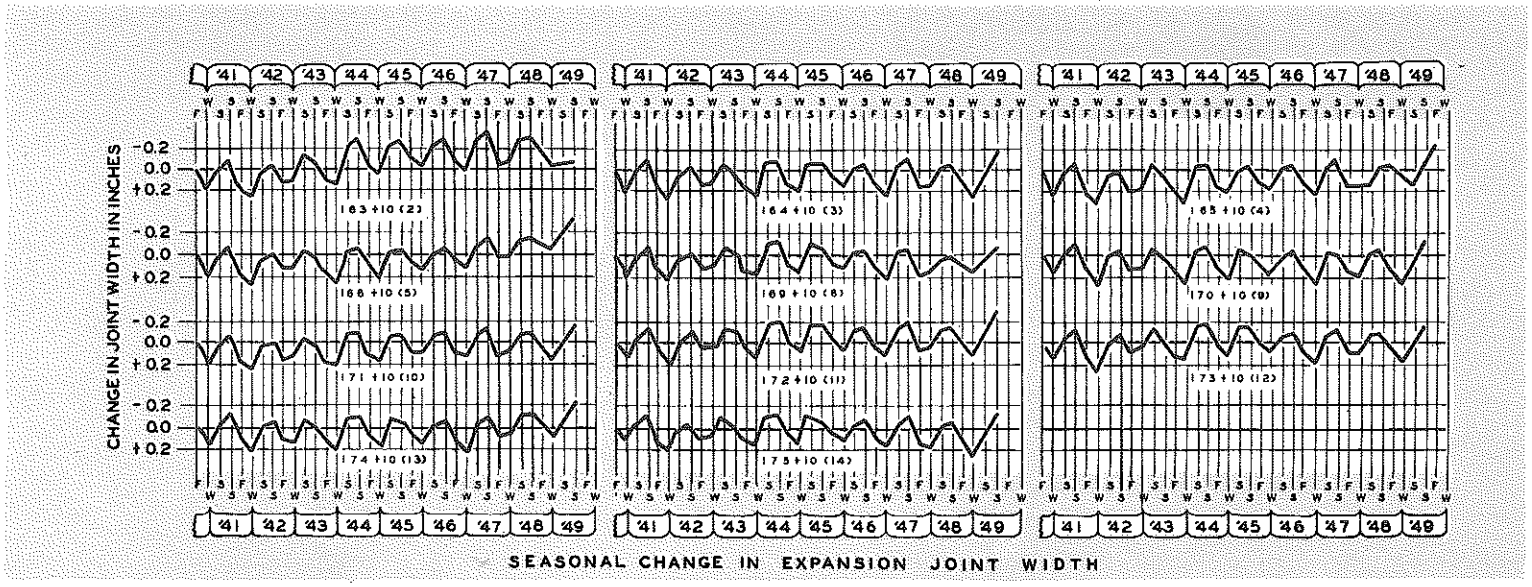


A. Joint construction.
B. Pressure cell in place.
C. Condition of joint before sealing.
D. Testing concrete specimens.

STRESS CURING OPERATIONS

Figure 45





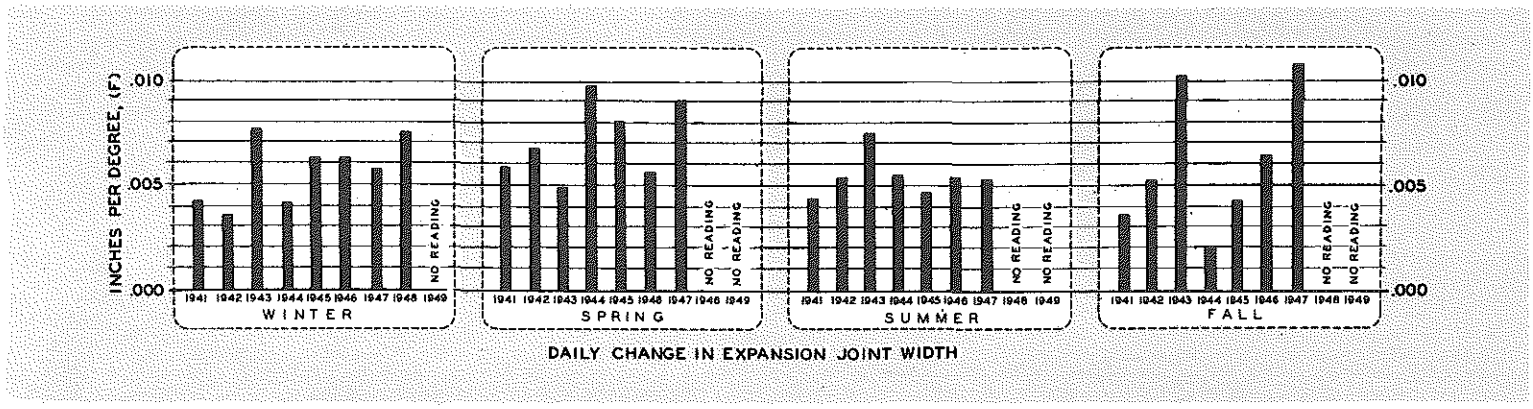


Figure 46

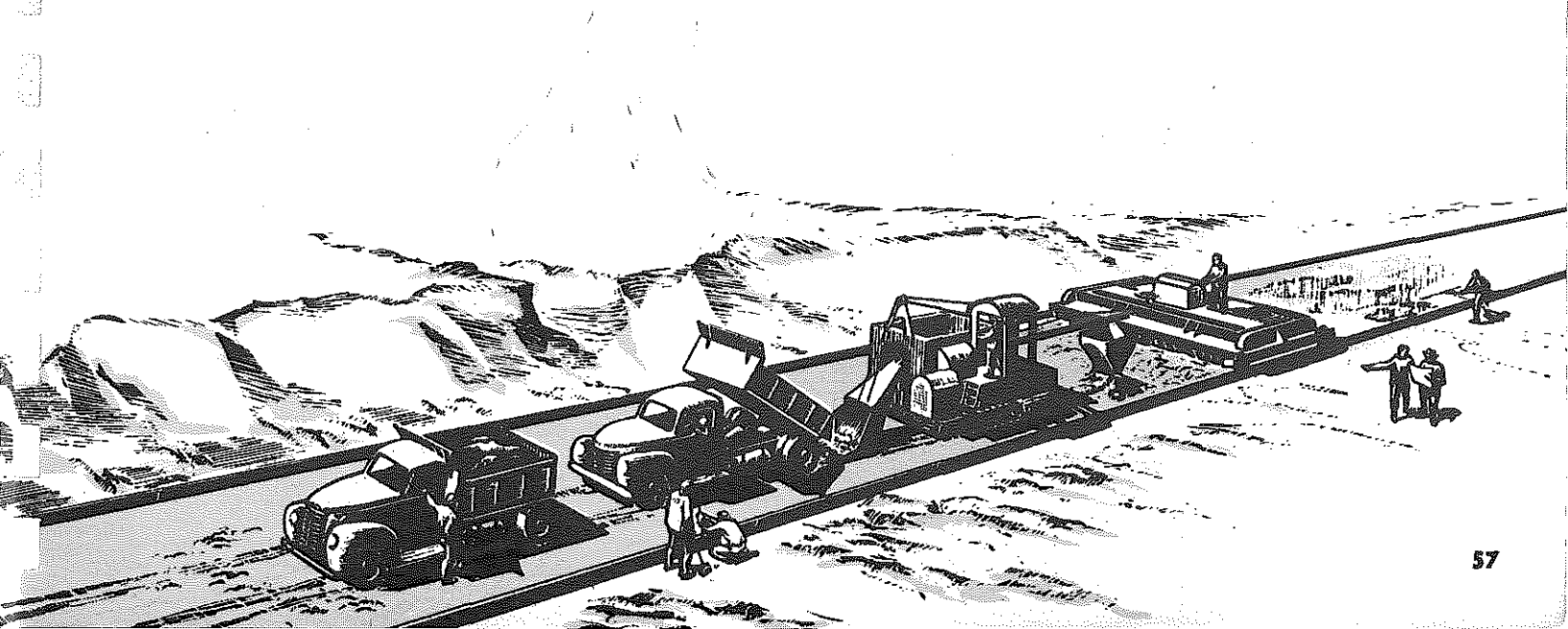
STRESS CURED CONCRETE

A careful analysis of the cracked slabs and subgrade has definitely proven that the cracking in four of the slabs can be attributed directly to abnormal changes in the subgrade caused by undesirable soil conditions and not to any factor or weakness in the slab structure due to method of construction. Part of Series 9A was constructed on a 12-inch sand subbase overlying a loamy sand with clay pockets and sandy clay loam soils. Apparently the 12-inch sand subbase was not thick enough, since there is evidence of rutting and intermingling of the subbase and subgrade materials causing improper drainage under the slab. The slab between Joints 15 and 16 is over a balance point between cut and fill and, therefore, cracking has resulted unquestionably from differential volume changes between the fill and cut section and has been augmented by poor subgrade soil characteristics. The cracks in the slabs between Joints 16 and 17, and 19 and 20 were also caused by poor subbase and subgrade conditions.

SLAB MOVEMENT: The maximum and minimum joint width movements for the winter and summer seasons from 1941 to 1949 are also shown graphically in Figure 46. The character of the graphs indicates: *first,* the joint width movements of the sound slabs are very similar in character both with respect to amounts and trends; *second,* the sound slabs seem to have a residual contraction which is evidently caused by the relatively high pouring temperature of approximately 80° F.; *third,* the movement of the joints at either end of slabs containing cracks or intermediate joints responds in the same manner as normal slabs with intermediate contraction joints in that the amplitude diminishes with time and a progressive residual displacement takes place at the slab ends adjacent to the expansion joint.

MECHANICAL SPREADING OF CONCRETE

With the exception of 600 lineal feet between Stations 764 + 00 and 770 + 00 of Series S, the concrete on the Design Project was placed and consolidated by means of a mechanical concrete spreader. The purpose of the study was to compare the relative merits of mechanical spreading and hand spreading of concrete with respect to economy to the user and quality of the finished pavement. It was proposed to make a comparative study of the uniformity of distribution and placing of concrete, concrete strength and characteristics, and the segregation of aggregates during placing and finishing.



At that time the mechanical spreading of concrete on road construction was not new, since it had been used in conjunction with side discharge truck mixers and with marked success on the Pennsylvania Turnpike. However, on the Michigan Test Road it was the first time that mechanical spreading was employed on 22-foot construction.

The mechanical spreader used on the Michigan Test Road was designed and constructed by the Jaeger Machine Company of Columbus, Ohio. The Jaeger concrete spreader is a power driven four-wheel frame with suitable forward and reverse speeds, on which is mounted a reversible 14-inch spiral screw which manipulates the concrete to one side or the other at the will of the operator. The screw may be quickly adjusted vertically so that the strike-off plate, which is located behind the screw, may be used to obtain the desired height of concrete on the subgrade. The concrete is dumped on the subgrade from the power bucket at regular intervals and the spreader screw moves through the pile breaking it down and distributing it laterally, first in one direction and then in the other, resulting in a uniform distribution of the concrete throughout the slab. At the same time, additional mixing of the concrete is obtained. The mechanical spreader was designed and constructed to handle the concrete for the full 22-foot slab width and to operate on the slab forms. Figure 47 illustrates the unit in position for operation.

MIXING AND DISTRIBUTION OF THE CONCRETE: When concrete is dumped onto the subgrade from the mixer bucket, varying degrees of segregation of the aggregates may be noted and the distribution of the concrete batch is never uniform. In the hand method of placing concrete, these conditions are not materially corrected. However, the thoroughness of mixing by the screw on the mechanical spreader not only of single batches, but the mixing of several batches together, apparently corrected these conditions and thereby produced uniformity in the pavement.

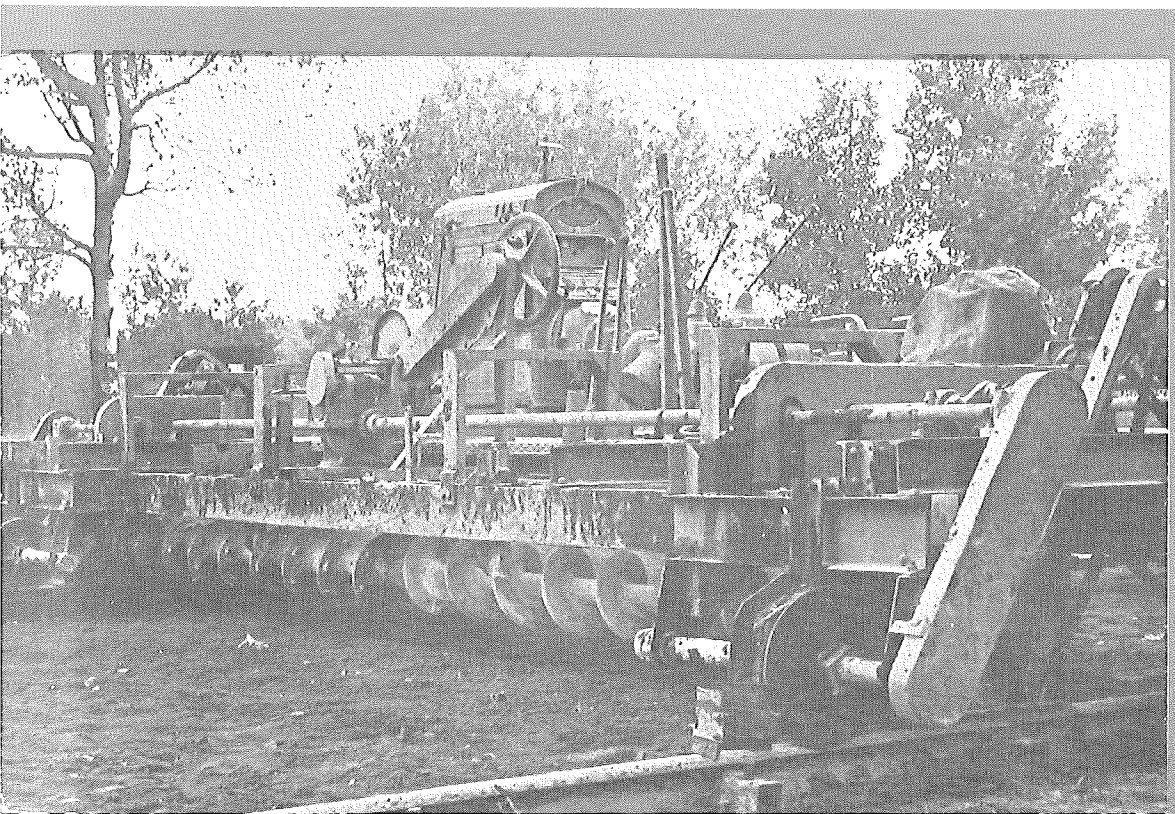


Figure 47A

MECHANICAL CONCRETE SPREADER

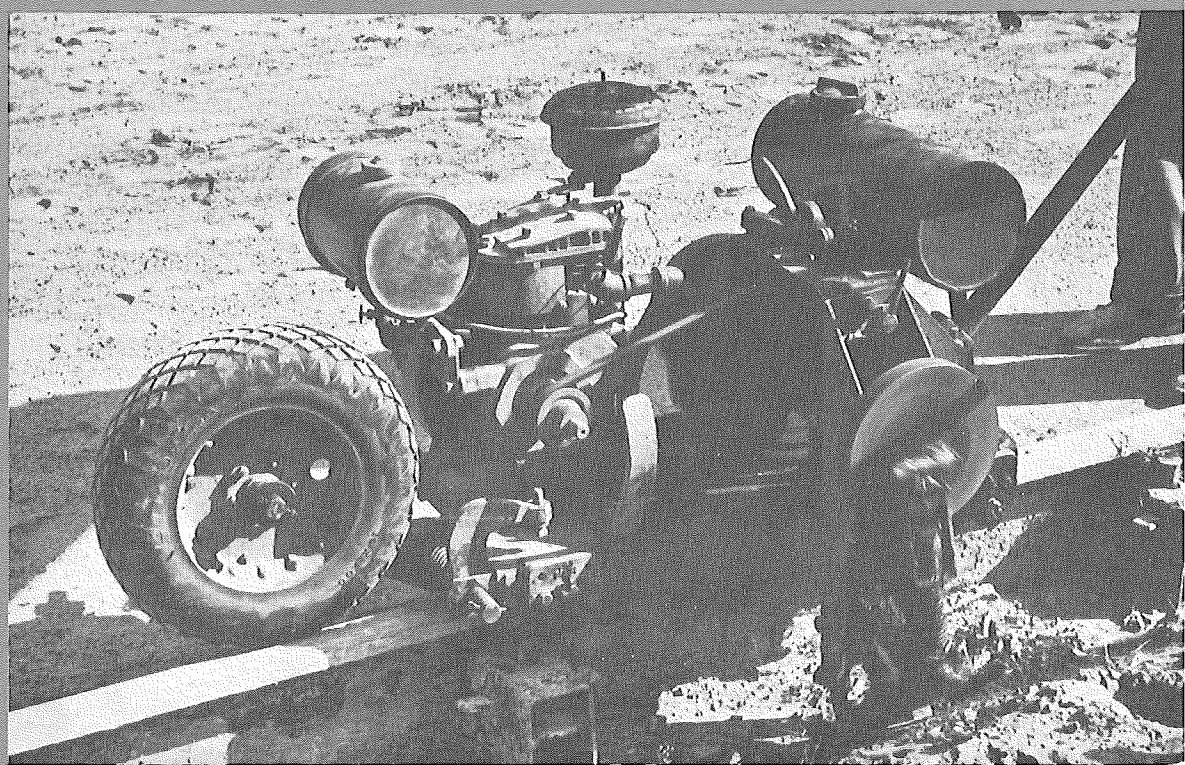
OPERATION: The success of the spreader depends upon the ability of the operator to handle the machine properly under all conditions. It was observed that in order to obtain desirable results from the spreader, it was necessary to coordinate the forward motion of the machine and the rotating speed of the screw under varying load conditions. When the forward motion of the spreader was too slow, the rotating screw fed too much concrete to the strike-off resulting in a bulging of the surface behind the strike-off. When the forward motion of the machine was too rapid for the rotational speed of the screw, a comparatively smooth but loose surface was formed; the surface had a tendency to tear and ravel. Under normal operation of the spreader, when the ratio of the forward motion of the machine was synchronized with the rotational speed of the screw under normal uniform load, a smooth and compact surface resulted.

The equipment was definitely too heavy for the weight of road forms used on this project. During operation the thrust of the screw was sufficient to cause considerable sidewise movement of the forms and, in certain instances, permanent damage. There was no indication, however, from subsequent measurements that the movement of the forms created surface irregularities which exceeded specification tolerances.

ECONOMIC CONSIDERATIONS: On this project it was problematical whether or not the spreader delayed operation to the extent that it affected the number of lineal feet of pavement which could be placed in a day. It was true that the mixer was delayed a portion of the time in the process of strike-off for the mesh; however, no study was made of the time saved by the mixer in being permitted to dump batches almost at will and with little regard to location, which cannot be done under standard practice. Even allowing that a measurable amount of time was lost with no compensations, this time is only a fraction of the time losses suffered in various other operations.

Figure 47B

MECHANICAL FORM TAMPER



The data obtained from comparative strength tests and analysis of fresh concrete samples were insufficient to furnish conclusive evidence as to the advantages of mechanical spreading versus hand spreading from the standpoint of producing uniform concrete.

MECHANICAL TAMPING OF FORMS

A mechanical form tamper was specified for use on the Design Project for the comparative study of mechanical versus hand methods of tamping forms on granular subbases and subgrades. The particular type of mechanical tamper employed is illustrated in Figure 47.

The mechanical tamper proved so superior to hand tamping that the Department now requires its use on all concrete pavement construction.

BITUMINOUS-RUBBER JOINT-SEALING COMPOUNDS

Three types of bituminous-rubber joint-sealing compounds were used on the Design Project for comparative study in conjunction with joint design. These materials consisted of two types of asphaltic oil-latex compounds developed by the Department and a commercial type of hot-poured rubber-asphalt compound known as Thermoplastic No. 52144.

One type of asphalt-latex compound, designated Type 1, consisted of a mixture of 70 parts of asphaltic oil SC-6A, 30 parts of normal rubber latex and 2 parts of hydrated lime. The materials were mixed together under controlled conditions

Table VIII

SUMMARY OF JOINT SEAL DATA

Joint Condition*	Evaluation in Percent					
	Asphalt-Latex, Type 1		Asphalt-Vultex, Type 2		Thermoplastic, Type 5	
	1945	1949	1945	1949	1945	1949
Effective seal against water	30	0	25	0	33	33
Effective seal against dirt	33	30	33	30	33	33
Plasticity	18	5	20	5	31	16
Overall rating	81	35	78	35	97	82

*All three items used in evaluation have equal weight, so that a rating of 33 per cent for a given item indicates perfect condition for that item.

immediately prior to sealing the joint. The other type, designated Type 2, was identical with Type 1 except a commercial vulcanized latex known as Vultex was substituted for the normal rubber latex.

The Thermoplastic compound, designated as Type 5, was a hot-poured type commercial rubber compound furnished in block form which upon heating to 450° F. transforms to a liquid of proper consistency for pouring into prepared joints.

These materials were installed during the construction of the project as part of the contract. Their respective locations are given in Table I.

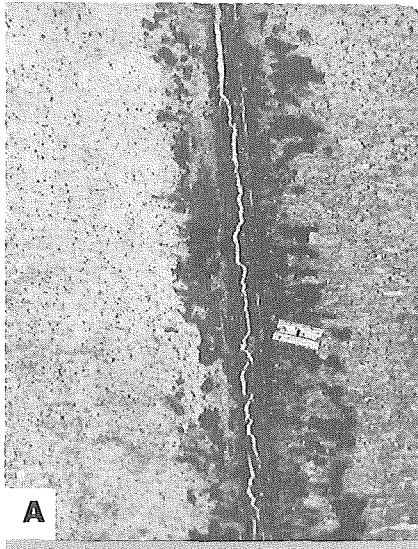
During 9 years of service, none of the sealing compounds has required maintenance at any time. The materials have weathered in varying degrees, however, and have become considerably more inspissated with age.

Condition surveys made in 1945 and 1949 revealed a measurable difference in service behavior of the three products as shown in Table VIII. The condition rating values given in Table VIII are based on the apparent plasticity and effectiveness of seal against water and dirt as manifested by the degree of cohesion and bond failure evident in each joint.

The asphaltic oil-latex compounds have reverted to a putty-like consistency, losing all their original plasticity. Permanent cracks have formed in the materials. See Figure 48 A and B.

The Thermoplastic hot-pour rubber-type compound is still in excellent condition after 9 years of service. See Figure 48 D.

The study definitely indicates that the new rubber-type joint seals are superior to the straight asphalt or tar products commonly used for sealing joints and cracks in pavement.

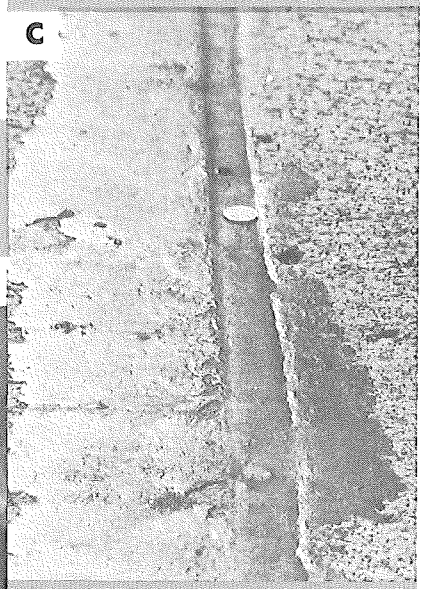


A

Typical condition of asphalt-latex joint seal materials in winter (1945).

B

Typical condition of asphalt-latex joint seal materials in summer (1949).

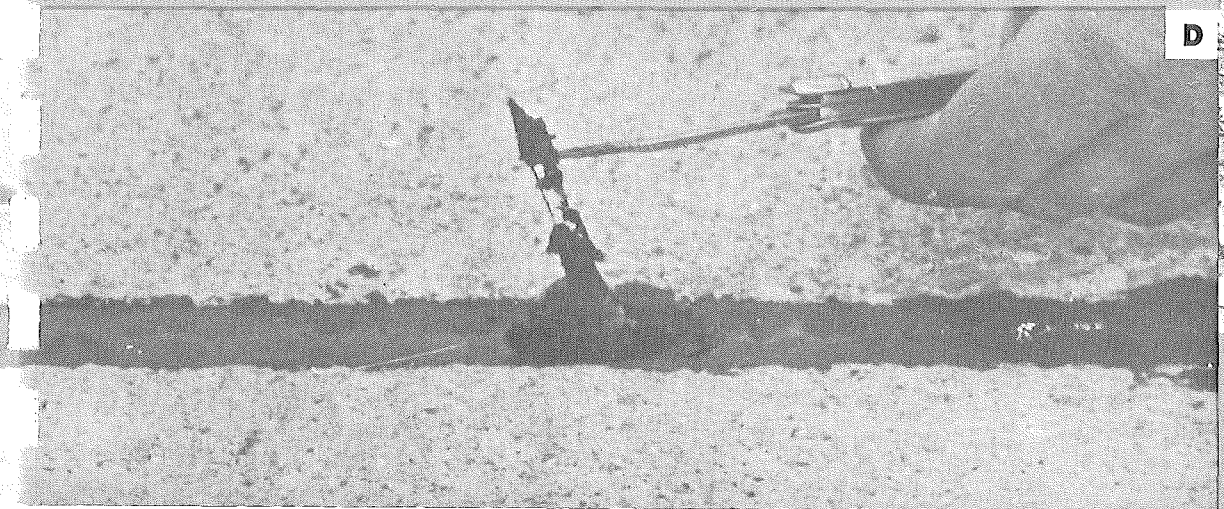


C

Typical condition of Thermoplastic sealing compound after 9 years.

Figure 48

Consistency of asphalt-latex materials after 5 years (1945).



D

PAVEMENT ROUGHNESS

In September 1941, and again in August 1949, a series of pavement surface roughness tests were conducted on the entire Test Road by personnel of the Bureau of Public Roads using their specially designed machine constructed to record the number of surface irregularities in a definite distance. See Figure 49. The study was made primarily to compare the riding qualities of various sections of the pavement, especially of those having varied expansion and contraction joint spacing, and to determine change in roughness with time.

In conducting the tests each section of each series was taken as an increment to compare the surface roughness condition for the different joint spacings and concrete mixture variations. In 1941, additional tests were made on nearby projects, one of the same age as the Test Road and another project 20 years old, to afford a comparison with standard construction and to determine an expectant roughness factor. The roughness data obtained from the two series of tests are presented graphically in Figure 50.

Considering first the results of the 1941 roughness tests, it is indicated that in the early life of the Test Road the roughness factor has no significant relation to joint spacing, especially when good workmanship is attained. It is evident that Series 3 and 4, with 20- and 10-foot contraction joint spacing, have slightly lower average roughness factors than those of Series 1 and 2 with greater contraction joint spacing. The roughness factor for Series 5, 6, 7, and 8 is slightly higher than that for the pavement in Series 1 to 4 inclusive. Series 9A, with 100-foot slabs and no intermediate joints, has a roughness factor of 80, which is below average for all of the other series. In Series 10, Sections A1 and A2 with dowel bars have a materially lower roughness factor than Sections B1 and B2 which were constructed

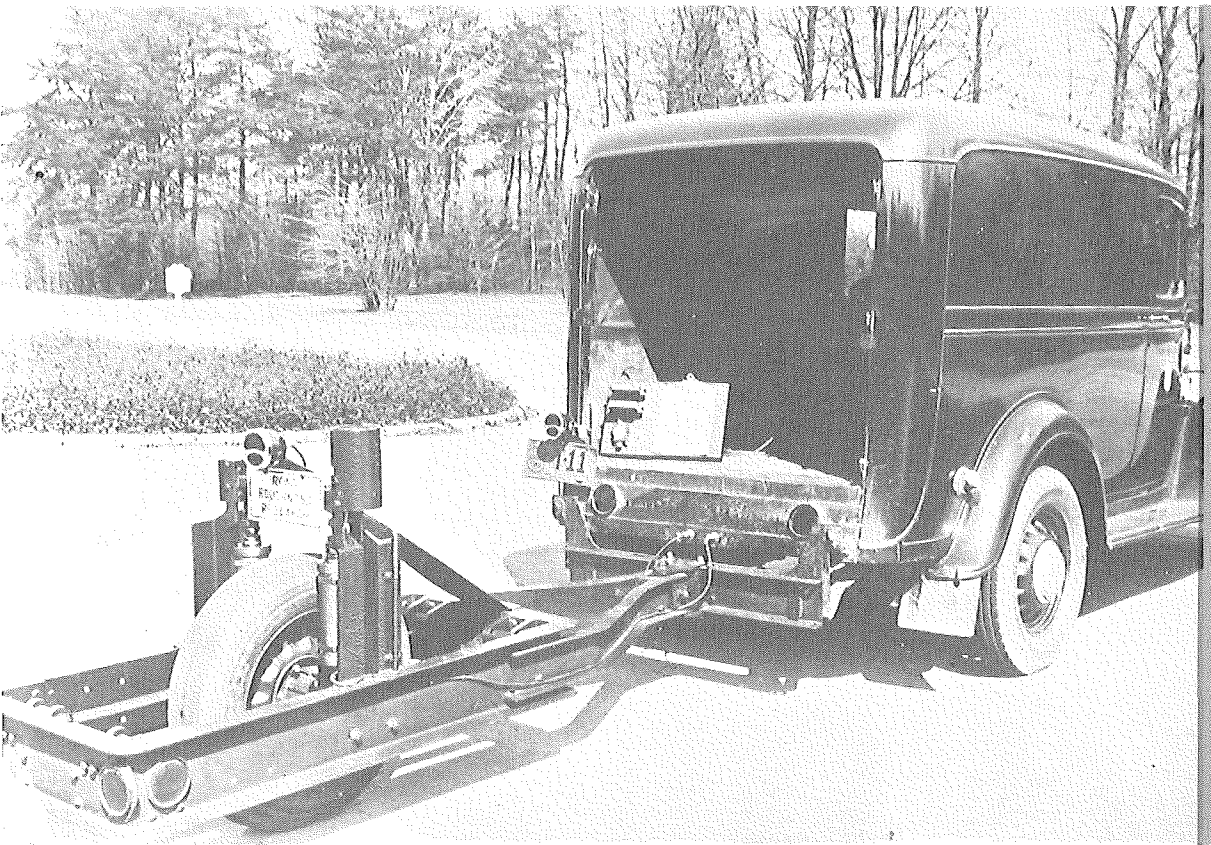


Figure 49

EQUIPMENT USED TO DETERMINE ROAD ROUGHNESS BY BUREAU OF PUBLIC ROADS

without dowels at transverse joints. Series 11 and 12 contain long, continuous slabs with and without reinforcement, respectively. Both series have a camparatively high roughness factor which cannot be attributed to the presence of transverse joints. However, Series 11, with the steel mesh, had a lower roughness factor than Series 12, without steel, which might be significant. Finally, the data indicate that the original roughness factor for the entire project ranged from 73 to 101 units per mile, an indication, in general, of good workmanship and excellent riding qualities.

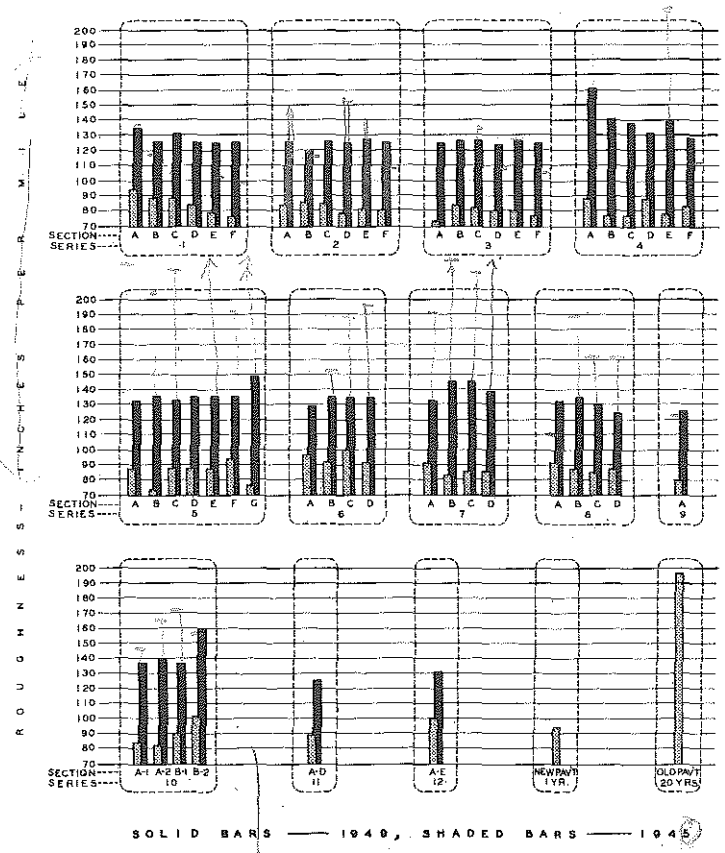
The 1949 roughness tests show that all sections have increased in roughness approximately to the same degree. However, Series 4 with the 10-foot joint spacing had the greatest increase in roughness. The general increase in roughness for the entire Design Project is approximately 56 per cent.

PAVEMENT AND SUBGRADE MOISTURE STUDY

Knowledge of prevailing moisture conditions in both the concrete slab and subgrade soil is necessary for an adequate appraisal of pavement behavior and is of considerable value in road design. In order to obtain a progressive record of the amount of moisture in the pavement and subbase of the Michigan Test Road, the electrical resistance method of Bouyoucos and Mick [3] was used. This method has the advantage of simplicity and rapidity of field determinations but is subject to definite limitations in some of its applications, some of which will be discussed in more detail later on. In spite of these disadvantages, however, a great deal of helpful information has been gained which may be applied to the interpretation of observed behavior of the Test Road. Since both the amount and distribution of moisture in slab and subgrade depend to a great extent on the characteristics of the soil underneath the pavement, some time should first be devoted to this phase of study to serve as a background for the analysis of observed moisture data.

Figure 50

ROAD ROUGHNESS DATA



SOIL CONDITIONS: The subgrade and subbase materials supporting the concrete slab in the Design Project are mostly composed of well-drained sandy or gravelly soils with the exception of two areas, from Station 88 + 00 to 129 + 00 and from Station 170 + 00 to 226 + 06, where it was necessary to construct a 12-inch sand subbase on the existing subgrade material. At other sections of the road, where the right of way passed through low lying areas of Saugatuck and Newton soil, suitable drainage of the subbase was obtained by filling to grade with sand borrowed from neighboring pits. Moisture cell and thermocouple assemblies were installed at four convenient points along the project so that temperature and moisture observations might be made concurrently with displacement measurements. The installations are located at Stations 772 + 10, 851 + 80, 1055 + 75, and 61 + 05. Samples of soil were taken from underneath the concrete pavement at each of these locations for standard tests and for calibration of the moisture cells. A summary of the physical properties of the four soils has been given in Table II.

In addition to the laboratory examination of the soil itself, there are several other factors to be taken into account when interpreting observed data. Such factors have to do with the particular features of location and environment which also largely influence moisture conditions at a given point. In Table IX are given pertinent data both on natural soil conditions and modifications due to construction procedure. These observations were made during the latter part of June 1944 following a half-year period of abnormally low precipitation and probably indicate a subnormal level of the water table; in exceptionally wet years the water table may rise somewhat higher.

METHOD OF MEASUREMENT: The method of determining moisture content by Bouyoucos and Mick consists essentially of measuring the internal resistance of a plaster of paris block under varying moisture conditions by means of a specially constructed wheatstone bridge. Each moisture cell consists of two bare wire terminals separated one inch and cast in chemically pure plaster of paris to form blocks ½- by 1½- by 2½-inches in size. Moisture equilibrium between block and surrounding medium is attained rapidly and the indicated resistance is a function of the amount of water present. Alternating current is used in the measuring bridge to eliminate the effects of electrolysis and polarization.

Table IX

SUB-GRADE SOIL CONDITIONS AT MOISTURE CELL STATIONS

Station	Soil Type	Profile	Water Table	Cut	Fill	Depth of Ditch
61+05	Grayling sand	0-13.5′ gravelly sand	13.5′	1.2′		2′
772+10	Rubicon sand	0-10.0 sand 10.0-12.0 gravel	12.0′	0.6′		2.5′
851+80	Newton sand	0-4.0 sand	3.5′		2.4′	4.5′
1055+75	Saugatuck sand	0-5.0 sand fill 5.0-5.5 loamy sand 5.5-7.5 sand 7.5-8.0 loamy sand	8.0′		4.8′	9.0′

*Note: See Table II for physical characteristics.

Most of the difficulties in the use of the method arise from the fact that the measured resistance of the plaster of paris block depends not only on the moisture content of the block itself but also upon the physical and chemical properties of the surrounding medium, particularly as they affect its electrical conductivity. This effect, which is due to the passage of stray currents outside the cell, is especially pronounced when the block is surrounded by an electrolyte, such as occurs when the cell is placed in concrete. The heterogeneous nature of the concrete mass enclosing the cell is an additional disturbing factor. Still another limitation lies in the fact that the resistance of the cell is affected only by the presence of water in its liquid state and is not sensitive to water in any other of its forms.

Because of these inherent characteristics, the moisture cells must be calibrated individually in conjunction wtih the particular material whose moisture content is to be measured. The procedure involves imbedding the cell in the specimen with known initial moisture content, allowing the specimen to dry out gradually, and taking resistance readings which correspond to subsequent moisture values. After each change in moisture content, the specimen should be hermetically sealed for at least 36 hours before observing the corresponding resistance in order to allow complete equilibrium to be attained throughout the specimen. From the series of values thus obtained the resistance-moisture function may be plotted. Such a curve is shown in Figure 51, which was derived from the calibration of a moisture cell in soil from Station 851 + 80. Soils from all four stations were of a sandy type, exhibiting similar properties, and characterized by a relatively sharp break in the resistance-moisture curve in the region of four to seven per cent moisture content.

A typical calibration curve for concrete is given in Figure 52. This curve breaks very sharply up-

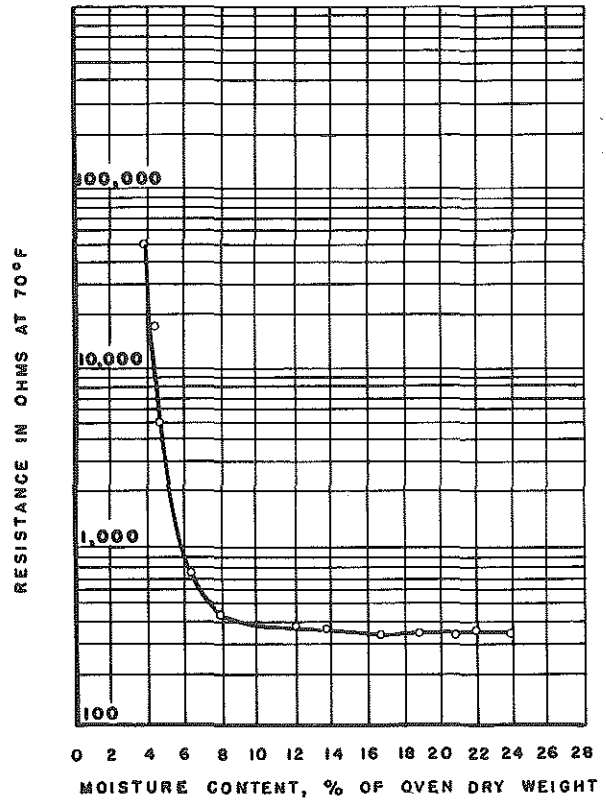
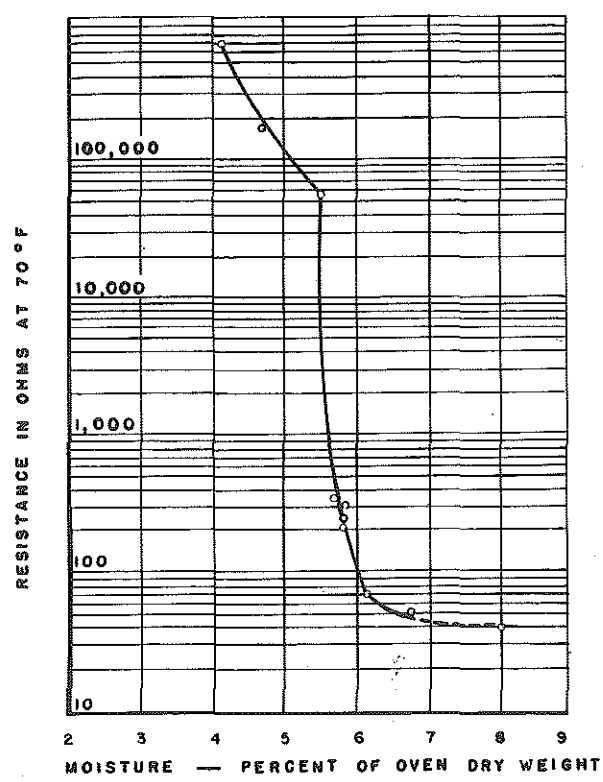


Figure 51

Laboratory calibration of moisture cells in soil. Newton sand.

Laboratory calibration of moisture cells in concrete.

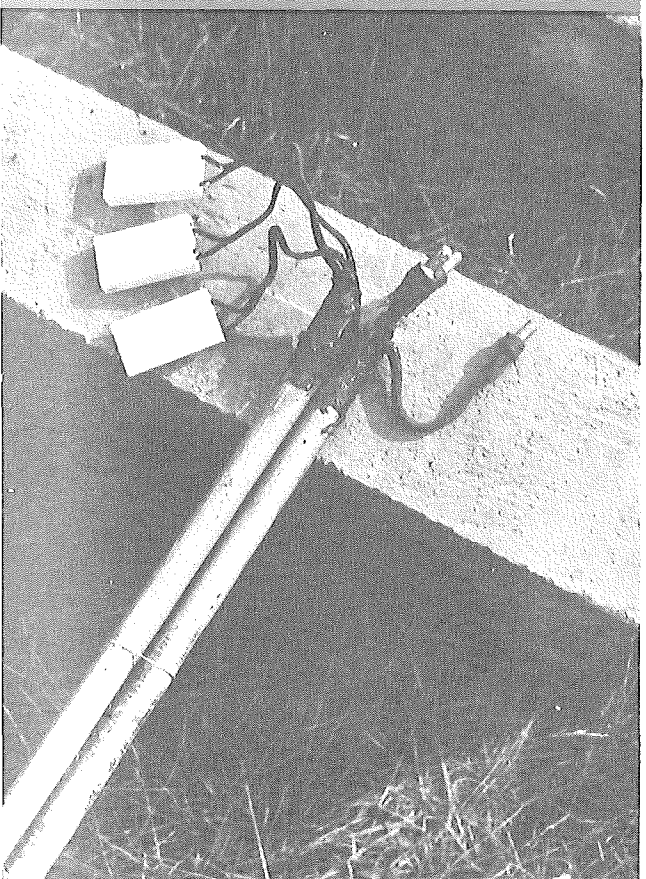
Figure 52



Electrical bridge assembly for determining moisture content.

Figure 53

Method of installing cells and thermocouples in concrete.



ward at around six per cent of moisture, the interval of 5.5 to 6 per cent representing a resistance range of approximately 40,000 ohms. All readings were corrected to a temperature of 70° F. in order to establish a common basis for comparison.

In the Design Project, the moisture cells were imbedded in the concrete at the top, middle, and bottom of the slab at different distances from the pavement edge, and in the subgrade at depths of one inch, six inches, and twelve inches below the pavement. Corresponding iron-constantan thermocouples were installed at the same time for temperature measurements. Photographs and details of these assemblies are shown in Figue 53.

MOISTURE DATA: All resistance and temperature measurements were first tabulated for each of the four installations separately in chronological order. After correcting each observed resistance to its equivalent at 70° F., the corresponding moisture content was read directly from the calibration curve for that location. In this way a series of moisture values was obtained for each location, for each season of the year, from the time of construction of the pavement up to the present time.

Subgrade moisture values are plotted in Figure 54 along with seasonal precipitation data. Only the data obtained from the moisture cells located 12 inches below the slab were used in the graph, since it is strongly suspected that the resistance characteristics of the two blocks immediately below the slab were appreciably affected by the overlying mass of concrete. Winter readings were also omitted since the method fails in subfreezing temperatures, although it may be used in conjunction with temperature observations to determine depth of frost penetration.

A summary of concrete moisture data also appears in Figure 54. Average seasonal moisture values are given for the top, middle, and bottom of the slab at the four stations of observation.

DISCUSSION OF RESULTS: The curves given in Figure 54 for moisture cell readings at the four stations show that the average moisture content of the subbase normally varies from about 4.5 to 6 per cent for the four locations observed, and that there is a reasonably close correlation with the seasonal precipitation. The data also indicate that the subbase is in general well drained, which condition would tend to maintain a relatively stable moisture content in the bottom of the slab throughout the year. These observations have been verified by periodic moisture determinations on samples taken from the subgrade of the four stations where temperature and moisture measurements were made.

It may be seen also that the moisture in the top of the concrete slab varies between about 4.7 and 5.5 per cent. In the middle the variation is somewhat less, moisture ranging from approximately 5.3 to 5.6 per cent, while the bottom maintains a fairly stable state at around 6.0 per cent total moisture content.

The overall moisture content of the slab was noticeably higher during the spring immediately following the construction of the pavement. During the first summer the bottom of the slab seems to have attained a relatively stable moisture equilibrium which has not changed appreciably with time. On the other hand, there has been a steady, though small, decline of moisture content at the center of the slab, amounting to about 0.4 per cent, from the initially observed values. It was to be anticipated that the top surface of the pavement would exhibit somewhat wider fluctuations in moisture content due to its direct contact with a more variable environment. In general, however, the vertical moisture differential in the slab was surprisingly small, for most observations amounting to less than one per cent. The effect of seasonal weather conditions on total moisture content of the pavement concrete was also less than might have been expected, although we know that the migration of moisture through a hardened concrete of good quality is extremely slow.

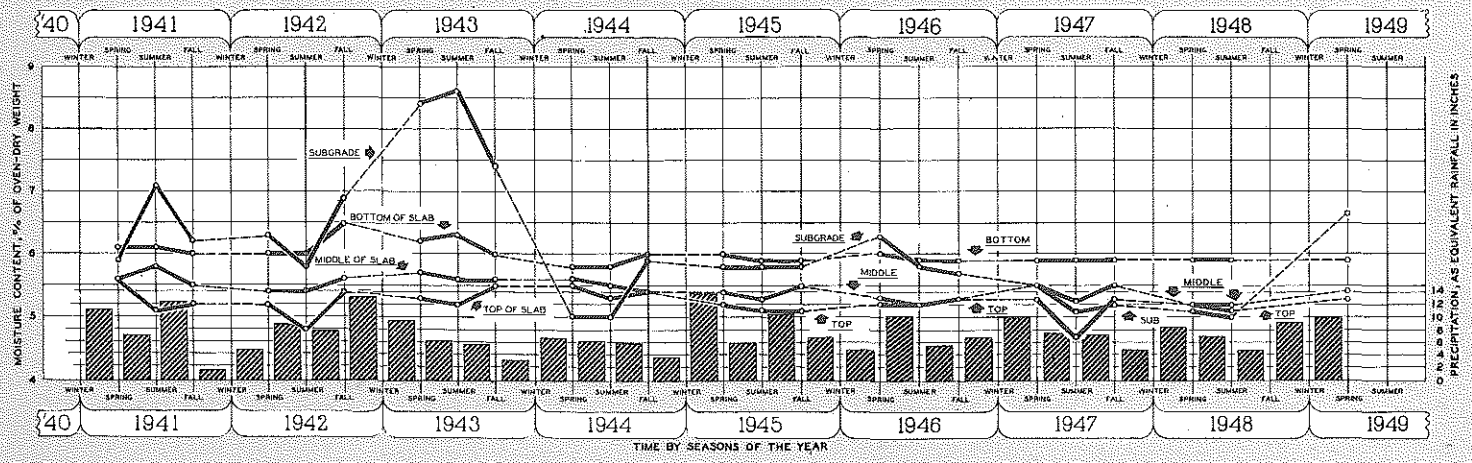


Figure 54

MOISTURE CONTENT OF PAVEMENT AND SUBGRADE

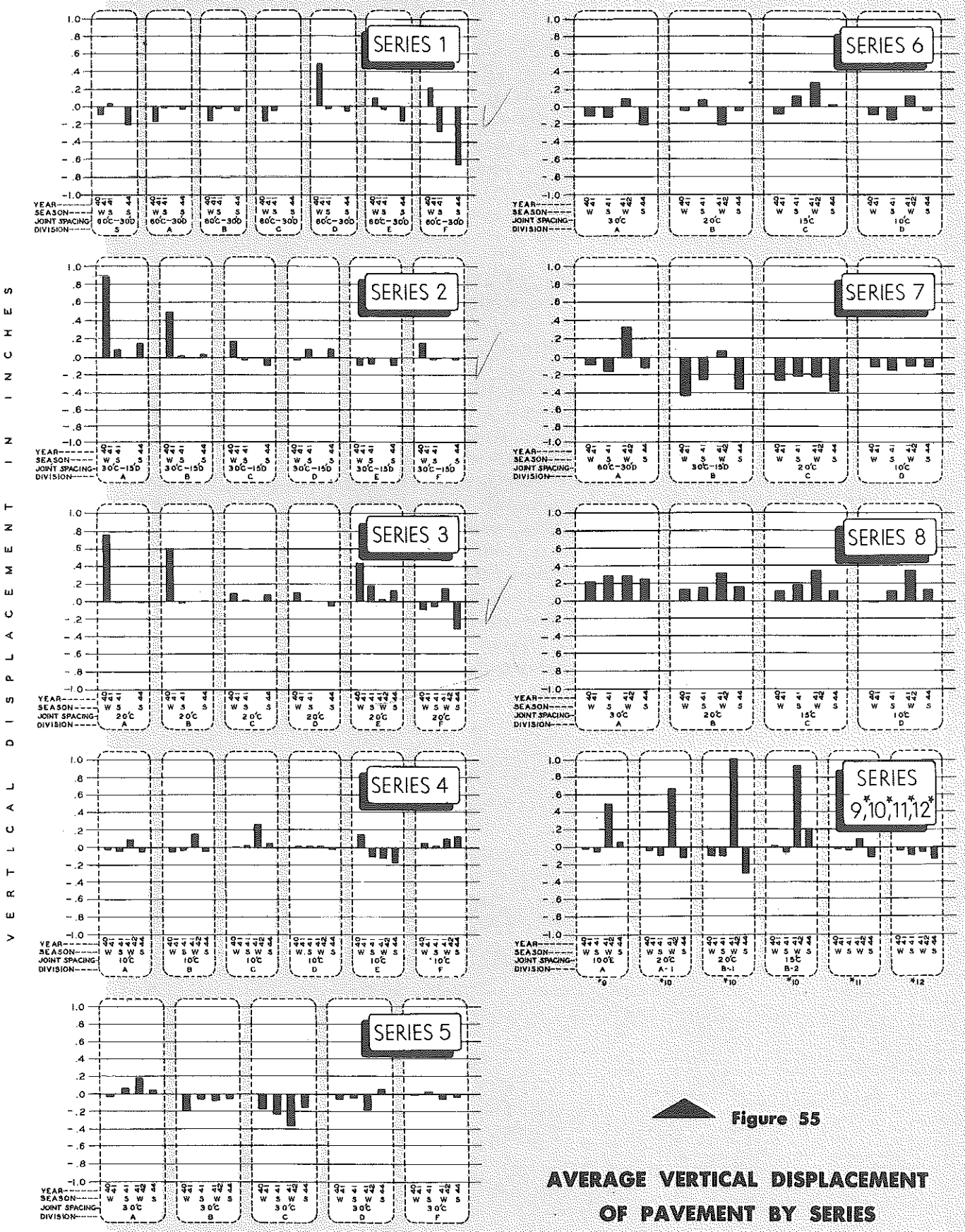


Figure 55

AVERAGE VERTICAL DISPLACEMENT OF PAVEMENT BY SERIES

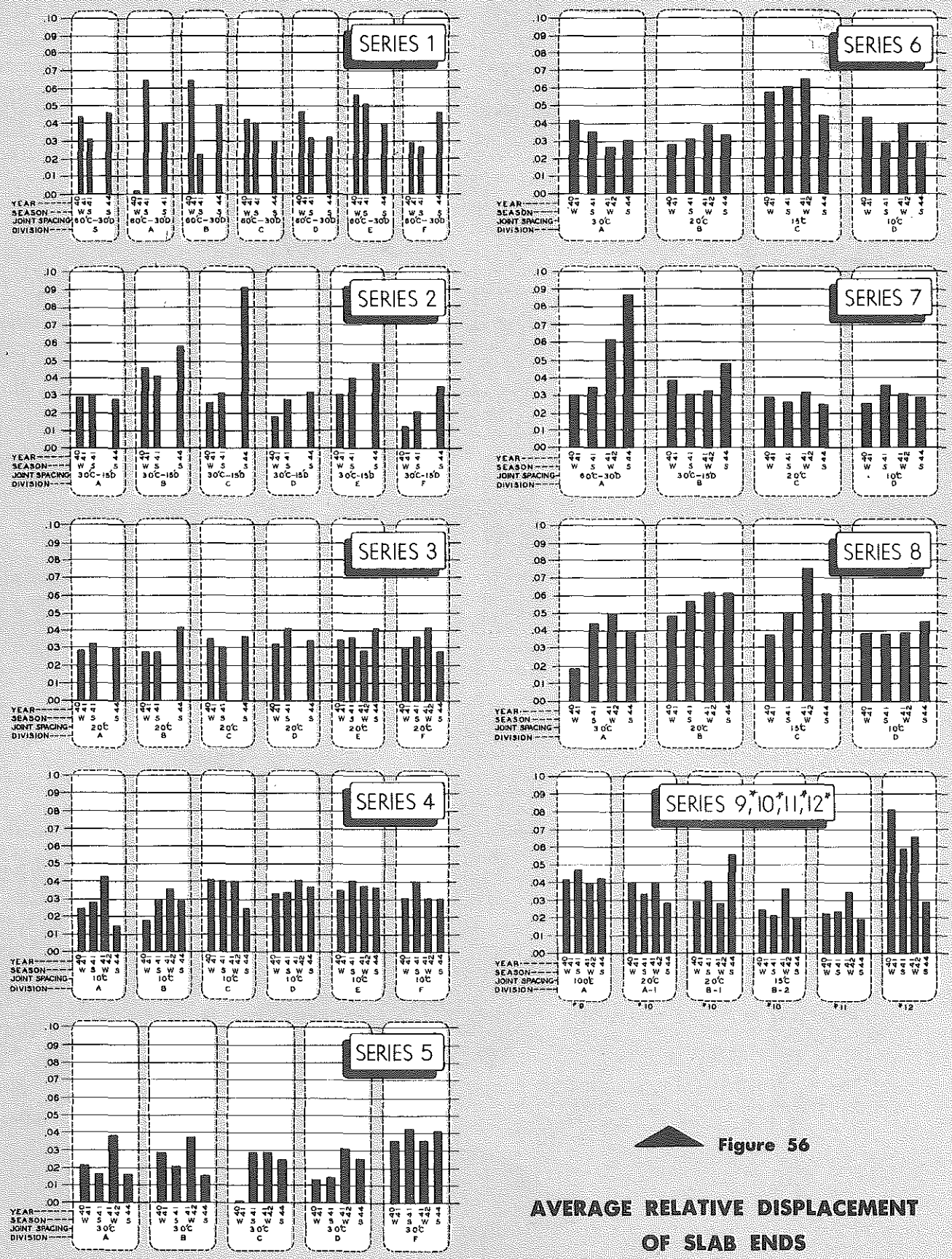


Figure 56

AVERAGE RELATIVE DISPLACEMENT OF SLAB ENDS

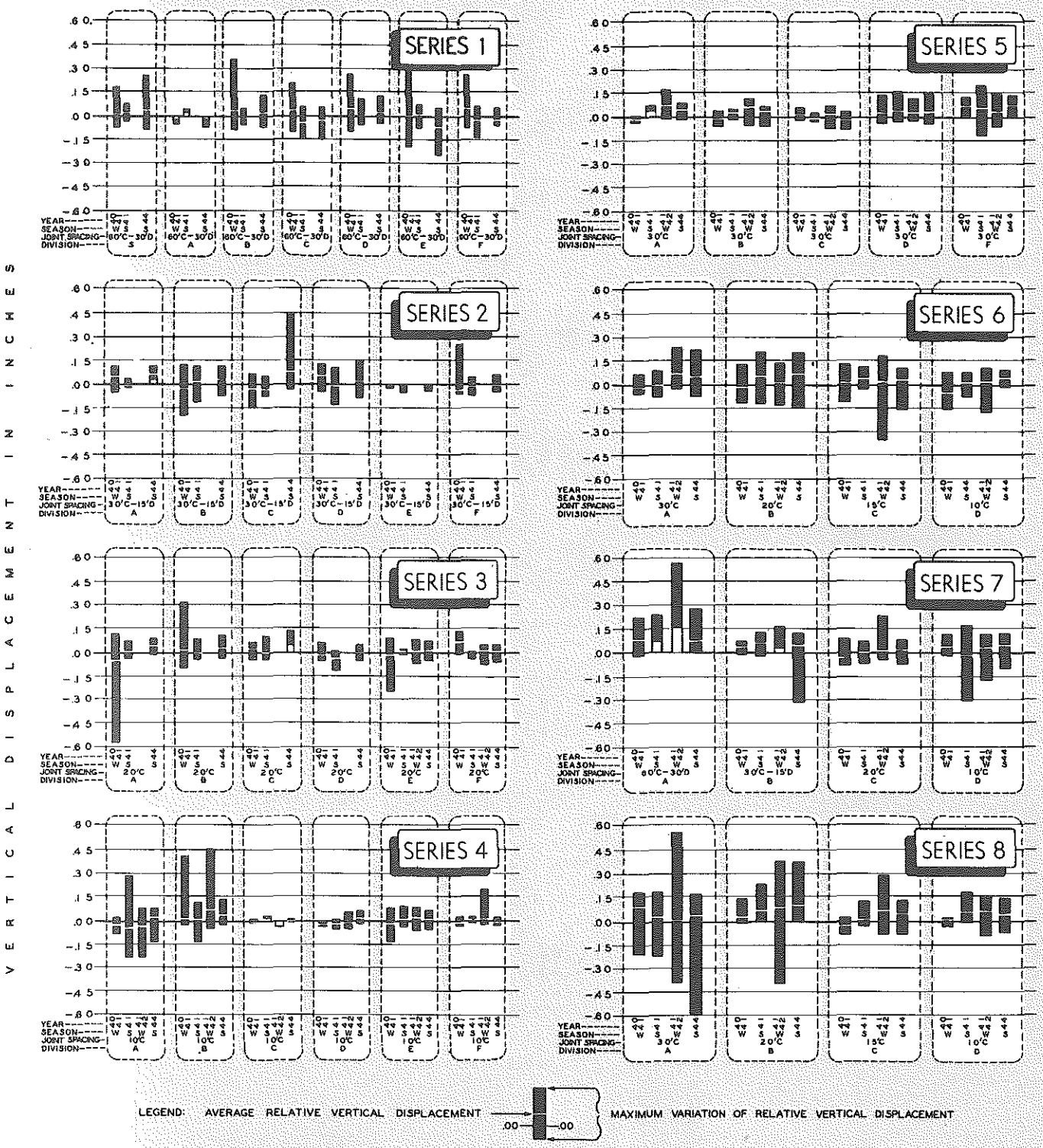


Figure 57

RELATIVE VERTICAL DISPLACEMENT OF SLAB ENDS WITH RESPECT TO CENTER

VERTICAL DISPLACEMENT OF PAVEMENT

During the 9-year period covered by this report, three sets of precise elevation measurements have been made over the entire length of the design experimental project. Level measurements representing pavement behavior under winter conditions were taken in 1941 and 1942. Summer level measurements were made in August 1941 and July 1944. All elevations are compared to the base readings which were established soon after construction of the project in 1940.

In general, the data in Figure 55 show that the average vertical displacement of the pavement throughout the Test Road has not exceeded 1 inch and in most cases it is less than ½ inch. However, in localized areas changes in elevation of as much as 1.95 inches occurred during the winter season, and were evidently caused by heaving. Extreme displacements are not shown in Figure 55. Some permanent settlement has occurred, ranging on an average less than ½ inch. In some cases the pavement has raised permanently between 0.2 and 0.4 inches.

Data showing the average relative displacement of slab ends, or faulting at joints, are presented in Figure 56. The maximum average faulting for the entire Test Road is less than ⅛ inch. A majority of the sections show faulting of less than 1/16 inch or, for all practical purposes, zero.

Data on vertical displacement of slab ends with respect to slab center, or in other words slab warping, are presented in Figure 57. The maximum variation of relative vertical displacement is shown as well as the average. The data indicate, in general, that many of the pavement slabs have attained a slight permanent upward warping while others have warped permanently downward.

No attempt has been made at this writing to correlate the displacement phenomena with design features. It is believed that the pavement is far too young at the present time to produce reliable information on performance in relation to certain design features incorporated in the Michigan Test Road.

STRAIN MEASUREMENTS

Strain measuring equipment was installed in the concrete at selected locations throughout a major portion of the Design Project for the purpose of analyzing slab stresses and volume changes. Interior strains at the neutral plane were measured by means of Carlson electric resistance strain meters. Surface strains were determined by the use of an 8-inch Berry strain gage on inserts set directly over the electric strain meters, and at various other locations.

Most of the interior installations consisted of a single meter placed longitudinally in the center of the north lane, 5½ feet from the edge of the pavement. At three stations, 772 + 11.5, 851 + 80, and 1055 + 75, groups of three meters were installed, one longitudinally at the center of the lane, another longitudinally at the edge of the pavement, and a third transversely at the edge.

The data from these observations have not been fully analyzed and interpreted as yet. However, preliminary computations and tabulations of the data indicate a very great influence of moisture on volume change and on the nature, distribution, and magnitude of stresses in the slab. Variations in moisture content of the subgrade, at Station 851 + 80, presumably due to proximity of the water table at that location (See Table IX), are reflected in very wide fluctuations of volume and stress indicated by the meters placed there. At the drier locations, 772 + 11.5 and 1055 + 75, the fluctuations are much less marked.

The compilation and analysis of the large volume of data obtained during the past 9 years is being continued and should yield further interesting information on pavement stresses and reactions.

GENERAL PAVEMENT CONDITION

Some physical changes of rather broad significance have begun to appear in the pavement of the Design Project. The area at the beginning of the project between Stations 764 + 00 and –3 + 65.4, which includes Series 1, 2, 3, and Section F of Series 4, is in excellent condition with very little evidence of scaling or spalling at the joints. The remainder of the project, however, is characterized by a quite general scaling of the concrete surface, with marked localization at the joints in many instances. This latter part of the project was built first and observations during construction plainly indicate poorer workmanship and wetter concrete here than in the remainder of the project.

Incidence of cracking in the first four series is following a course pretty much to be expected on the basis of the slab lengths involved. In Series 1, seven full transverse cracks and nine half cracks have formed. In Series 2, one complete crack and four half cracks have appeared. No transverse cracks have shown up in Series 3 and 4 so far.

There are five complete transverse cracks in Series 10, two of which are in the doweled sections and three in the undoweled sections. Cracks and other defects occurring in the pavement of Series 5 through 9 have been discussed earlier in this report.

GENERAL SUMMARY

The investigational work associated with the Design Project of the Michigan Test Road has produced to date several results of outstanding importance in the design and construction of concrete pavements. All of these findings which are recapitulated below have been utilized in framing the Department's current specifications for concrete pavement construction.

1. The satisfactory performance of long sections of pavement under full restraint indicates that expansion joints are unnecessary except at such places as intersections, rail crossings, and structures, where excessive compressive stresses introduced by expansion forces are undesirable.

2. Adequate load transfer devices are essential in all joints to preserve the mutual elevation of the abutting slabs.

3. The method of forming a contraction joint by grooving the surface of the pavement and subsequently filling the groove with a good sealing compound is far superior to the method of inserting a premolded bituminous fiber strip.

4. Continuous plate dowels are not satisfactory as load transfer devices.

5. The commercially available asphalt-rubber joint-sealing compounds have greater durability than the mixed-on-the-job asphalt-latex mixtures and are far superior to the straight asphalt or tar products in common use for sealing joints and cracks.

6. Mechanical tamping of soil under forms produces more positive and uniform results than hand tamping.

7. Nothing has been learned which would definitely indicate that short slab construction is superior to long slab construction and many advantages are to be gained by the latter practice by way of better riding qualities, lower maintenance costs, and better construction conditions.

8. Results so far indicate that the uniform cross section is equal in performance to that of the thickened edge section, with many obvious advantages.

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