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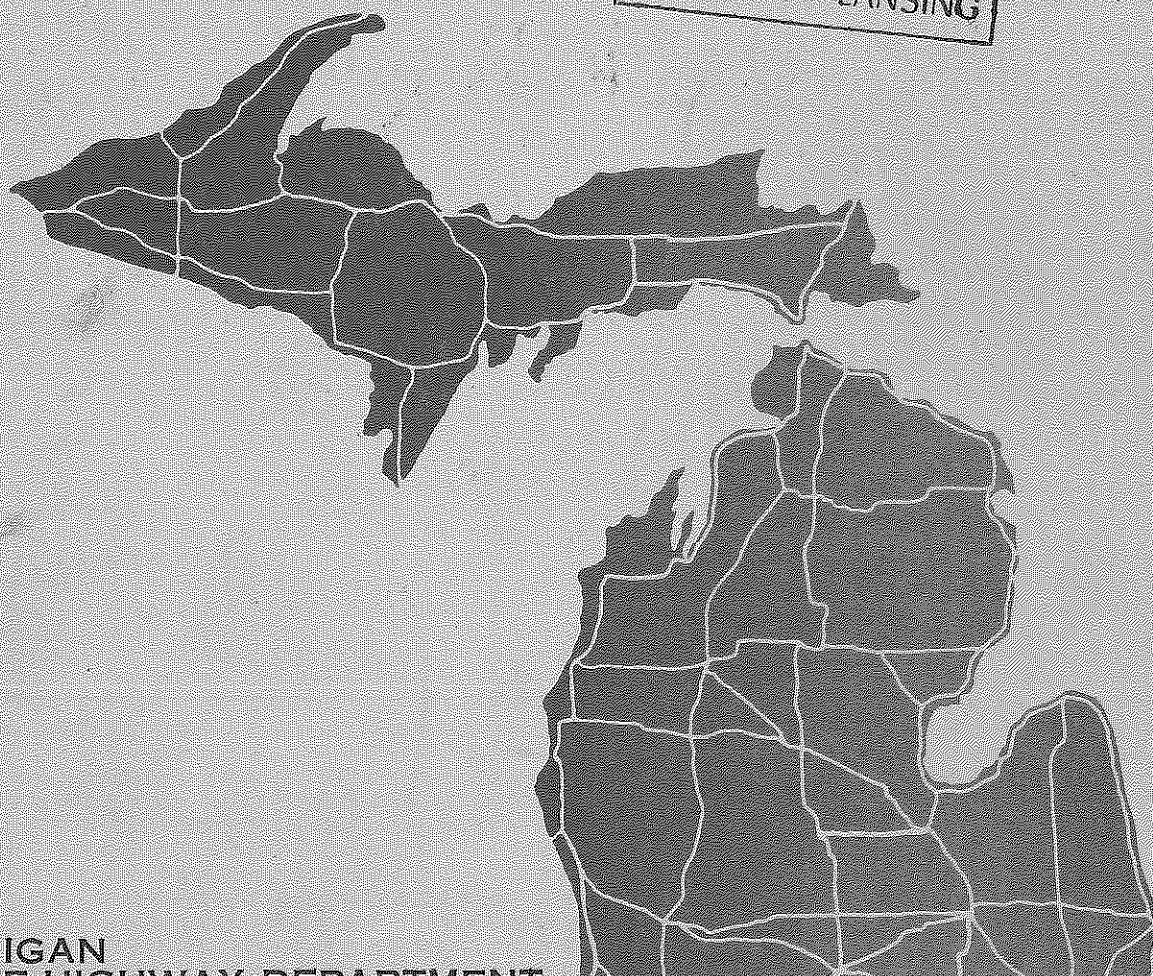
THE DESIGN OF CONCRETE PAVEMENTS FOR POSTWAR CONSTRUCTION



MICHIGAN STATE HIGHWAY DEPARTMENT

CHARLES M. ZIEGLER
STATE HIGHWAY COMMISSIONER

TESTING & RESEARCH DIVISION

Stokstad
1954

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MICHIGAN
STATE HIGHWAY DEPARTMENT
Charles M. Ziegler
State Highway Commissioner

THE DESIGN

of

CONCRETE PAVEMENTS

for

POST WAR CONSTRUCTION

Research Laboratory
Testing and Research Division
March 15, 1945

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

INTRODUCTION

The concrete pavement design presented in this report has been made with due consideration to all known requirements for a modern highway and is based on principles established by theory, research and experience. The recommendations contained herein have been prepared for consideration in connection with Michigan's post-war highway construction program.

The design as developed in the report recognizes four distinct classes of highways based on traffic conditions and embodies other features in pavement construction in keeping with the modern trend. The design features are briefly summarized as follows:

1. Highway Classes: I Expressway, II Heavy Primary, III Light Primary and IV Secondary.
2. The pavement will be of uniform thickness and properly reinforced.
3. Slab thickness according to class is respectively 9-10, 9-10, 7-9 and 6-8 inches.
4. Slab length between breaks in reinforcement is 100 feet. No intermediate joints.
5. Weight of reinforcement per 100 square feet for each thickness will be approximately 98, 98, 85 and 78 pounds respectively.
6. Expansion joints will not be installed during summer construction except at structures, intersections and at other locations specified. During fall construction, commencing September 15th, expansion joints will be placed at intervals of not less than 400 feet or not greater than that of one days pour as directed by the Engineer.
7. Wood boards one inch wide will be used for expansion joint filler.
8. Contraction joints will be installed at 100 foot intervals.
9. Contraction joints will be formed by a groove 1/2 inch wide and 2 inches deep in the surface, and in addition, a metal parting strip 1 inch high will be installed at the bottom of the joint.
10. A galvanized metal shield will be required at the bottom and edges of joints to prevent infiltration of foreign matter. It will be fabricated in conjunction with the dowel bar assembly.

11. Dowel bars, 1-1/4" x 15" spaced at 12 inch centers will be required at all transverse joints for the prevention of faulting.

12. All joints will be sealed with an asphaltic-latex joint sealing compound.

The selection of this design was influenced by the facts that: (1) every joint is potentially a source of structural weakness contributing to the ultimate deterioration of the pavement accompanied by increasing maintenance costs, (2) it is possible to design a concrete pavement slab up to 200 feet and possibly more in length of sufficient strength to withstand combined warping, load and friction stresses for any predictable traffic condition and subgrade support satisfying certain assumptions; (3) the service performance of 100 foot slabs constructed ten or more years ago on suitable subgrades has been very satisfactory. It would appear then to be good engineering practice to reduce the number of joints to an absolute minimum consistent with sound design practices and economic considerations.

The report has been presented in three parts. Part I explains in detail the method of determining cross-section thickness by stress analysis taking into account the principle of fatigue. Part II presents a discussion of several important factors which influence transverse joint spacing substantiated by theoretical considerations and the results from observational studies, including slab movement measurements on the Michigan Test Road. Part III covers the design of transverse joints including provision for slab movement, the exclusion of foreign matter and the design and spacing of load transfer devices. Detailed conclusions and recommendations are presented at the end.

For convenience to the reader, it has been thought advisable to preface the main body of the report by a summary of assumptions on which the design is based.

ASSUMPTIONS CONSIDERED IN THE DESIGN

1. The load and warping stresses are evaluated by the H. M. Westergaard theory with all its assumptions. The corner stresses are evaluated in the case of upward warping by the Public Roads Administration experimental formula and the corner temperature stresses by the formula proposed by Bradbury (See Text).

2. Slab thickness computations have been based on the assumption that the combined load, warping and friction stresses do not exceed values sufficient to produce cracks due to fatigue before 20 years for the traffic under consideration. This 20 year period, referred to as "crack expectancy", is believed reasonable and one possible to obtain under modern construction methods.

3. Westergaard's analysis considers loads applied in the interior, or at the edge, or corner of the slab. With the advent of wider pavements the possibility of all wheel loads reaching the extreme edge or corner positions is reduced. Thus, consideration has been given to both 100 percent and 50 percent wheel load applications at the critical points.

4. The subgrade is assumed to be uniform throughout, well drained, stable and to have a definite numerical value for subgrade reaction. The subgrade reaction "p" at a point is proportionate to the deflection at that point and to a coefficient "k" which is the same for all points of the slab and which is called the subgrade modulus.

5. Preliminary subgrade studies by the Department indicate that the modulus of subgrade reaction for 12 to 24 inch subbases of sandy or granular nature are in the order of 100 to 300 p.c.i. Since granular subbases are utilized in Michigan, the extreme values of 100 and 300 p.c.i. have been assumed for subgrade modulus "k" in computing slab thicknesses.

15" subbase = "K" of 200
24" " = " " 300

6. Laboratory and field studies by the Department and other highway organizations indicate that the subgrade friction coefficient varies between approximately 1.0 for light granular soils to 2.0 for heavier clay soils. A friction coefficient of 1.0 was used with "k" equal to 100 and a value of 2.0 with "k" equal to 300 for the determination of slab thicknesses.

7. The friction forces are assumed to produce only uniformly distribued direct tension or compression stresses without any bending of the slab.

8. The subgrade friction stresses for slabs of equal length and thickness are considered identical for both types of construction.

9. All slabs are considered to be unrestrained at the ends. Therefore, forces which may be caused by dowel bar friction, temperature or other sources are not considered in the analysis.

10. All joints are assumed to act as <u>ideal hinges</u> without the transmission of any bending moments and without permitting infiltration or affecting riding qualities.

11. Contraction joints containing load transfer devices are the only type of plane of weakness joints considered.

12. Expansion joints are planes of total slab separation in the pavement located at breaks in reinforcement. They usually contain a premolded filler material suitable for preventing infiltration and capable of absorbing compression forces. Slip dowels and other devices are generally included to preventing faulting of the slab ends.

13. For the type of pavement construction under consideration sufficient reinforcement is provided so that the width of transverse cracks, if they should appear, should remain very small in order to prevent the infiltration of foreign matter and to provide a certain amount of edge support. The whole friction force is assumed to be taken up by the longitudinal bars of the reinforcement without exceeding the yield point of the steel.

14. Under present construction methods dowels or other devices are considered necessary at expansion and contraction joints to prevent faulting only. Their ability to reduce edge stresses by load transfer is not considered in the design analysis, except in special cases when 50% load transfer features are provided. Consequently, an impact factor of 1.5 is utilized in computing slab thickness at transverse edges and at corners. At all other points the impact factor is considered to be 1.00.

15. Cracks occurring in reinforced pavement are assumed to permit 50% load transfer.

16. In the computation of warping stresses, the length of slab is considered to be the distance between any two consecutive joints.

17. The average interpolated slab thickness between the ones for ($k = 100$, $F = 1.00$) and ($k = 300$, $F = 2.00$) represents the thickness for $k = 200$, $f = 1.50$ for the same traffic and same crack expectancy.

18. Certain physical properties of concrete are assumed to have the following values in the analysis:

Modulus of Rupture	750 p.s.i.
Modulus of Elasticity	5×10^6 p.s.i.
Ultimate Compression	5,000 p.s.i.

Poisson's Ratio is 0.15 for loads and zero for temperature or warping stresses.

The Temperature Expansion Coefficient is 5.5×10^{-6}.

19. Four classes of traffic are considered defining the load forces.

20. The radius of load distribution is defined by Bradbury (See Text).

21. Temperature differentials of 3h in summer, 2h in winter and (h) at night are considered where "h" equals slab thickness.

PART ONE

DETERMINATION OF SLAB THICKNESS

The thickness requirements for the types of concrete pavements under consideration have been determined by stress analysis, since methods are available for calculating the stresses induced in a concrete pavement slab by external forces and temperature, and since the stress-resisting properties of the concrete material are well established.

In connection with the method of stress analysis, the principle of fatigue has been utilized as a means for analytically predicting the age at which cracks may be expected to occur in concrete pavements of different thicknesses, because it is possible to ascertain, with a reasonable degree of certainty, the particular traffic characteristics to which the pavement is to be subjected and the frequency of combined load, warping and friction stresses.

Special consideration has been given in the analysis to such factors as: wheel load frequencies based on actual traffic survey data, the latest recommendations of the Public Roads Administration in regard to stress analysis, the use of definite design values obtained from field and laboratory studies conducted by the Department and the introduction of an advanced method for determining the number of stress repetitions to cause failure.

The cross-section thicknesses for several types of pavement construction have been determined on the basis of combined load, warping and subgrade friction stresses employing the assumptions and data outlined early in the text.

In determining slab cross-section thickness, consideration is given to continuous slab lengths of 10, 20, 30, 40, 60, 100, 140 and 200 feet.

Traffic In Relation To Wheel Load Frequencies And Highway Classes

The daily frequency of wheel loads in excess of 4,000 pounds for the different classes of highways can be predicted with a reasonable degree of accuracy from existing traffic survey data. For the purpose of the design analysis, traffic data were obtained from the Planning and Traffic Division for the years 1936, 1942 and 1943. These data are presented in Tables 1, 2 and 3. Figure 1 is a graphical presentation of the data in Table 1. This graph has been used as a basis for estimating the future wheel load frequency for each class of load in terms of percentage of total daily axle loads occurring on certain routes representative of a particular class of highway.

The four highway classes recognized by the Design Division are:

I.	Express Way	- Divided Lane
II.	Heavy Primary	- 2 Lane
III.	Light Primary	- 2 Lane
IV.	Secondary	- 2 Lane

The classification of these types of highways by traffic characteristics is made in Table 4.

Determination of Critical Wheel Load Frequency. Tables 4 and 5 also contain the computed critical wheel load frequency per day and per year respectively. The daily wheel load frequency is determined by multiplying the percentage for each wheel class by total daily axle loads. The total critical wheel loads per year in Table 5 were obtained by multiplying the values in Table 4 by 365.

Load Stress Calculations

The magnitude of the stress developed in a pavement slab by a definite wheel load will be determined by considering: (1) The vertical component of the load, (2) The position of load with respect to edge of slab, (3) The area over which the load is distributed on the pavement surface, (4) Certain physical characteristics of the pavement material, (5) Certain physical characteristics of the subgrade, (6) Time duration of the load, and (7) Impact of load.

Formulas I, II, III and IV below are used for the calculation of stress values. These are derived from Westergaard's analysis (1)*.

Case I, for maximum unit stress at interior of slab (S_i)

$$S_i = \frac{1.1\ (1+u)}{h^2}\ P \left[\log_{10}\left(\frac{h}{b}\right) + 1/3\ \log_{10}\left(\frac{E}{K}\right) - 0.089 - 13.64\left(\frac{t}{L}\right)^2 Z\right], \text{ or}$$

$$= \frac{1.265\ P}{h^2}\left[\log_{10}\left(\frac{t}{b}\right) + 0.15831\right] \text{ - - - - - - - - - - - - - - - - I}$$

Case II, for maximum unit stress at edge of slab (S_e)

$$S_e = \frac{2.117 \times (1 + 0.54u)P}{h^2}\left[\log_{10}\left(\frac{h}{b}\right) + 1/3\ \log_{10}\left(\frac{E}{K}\right) - 0.2666\right], \text{ or}$$

$$= \frac{2.288\ P}{h^2}\left[\log_{10}\left(\frac{t}{b}\right) + 0.0897667\right] \text{ - - - - - - - - - - - - - - - II}$$

Case III, for maximum unit stress at Corner (S_c)

$$S'_c = \frac{3P}{h^2}\left[1 - \frac{a\sqrt{2}}{t}\quad 0.6\right] \quad \text{for downward warping - - - - - - - - III}$$

$$S''_c = \frac{3P}{h^2}\left[1 - \frac{a\sqrt{2}}{t}\quad 1.2\right] \quad \text{for upward warping - - - - - - - - IV}$$

*(1) Westergaard, H. M.: "Computation of Stresses in Concrete Roads". Proceedings of Highway Research Board, 1925. Part I.

GRAPHIC PRESENTATION OF WHEEL LOAD FREQUENCY BASED ON 1936 TRAFFIC COUNT

FROM REPORT TO PUBLIC ROADS ADMINISTRATION
BY PLANNING AND TRAFFIC DIVISION 1938

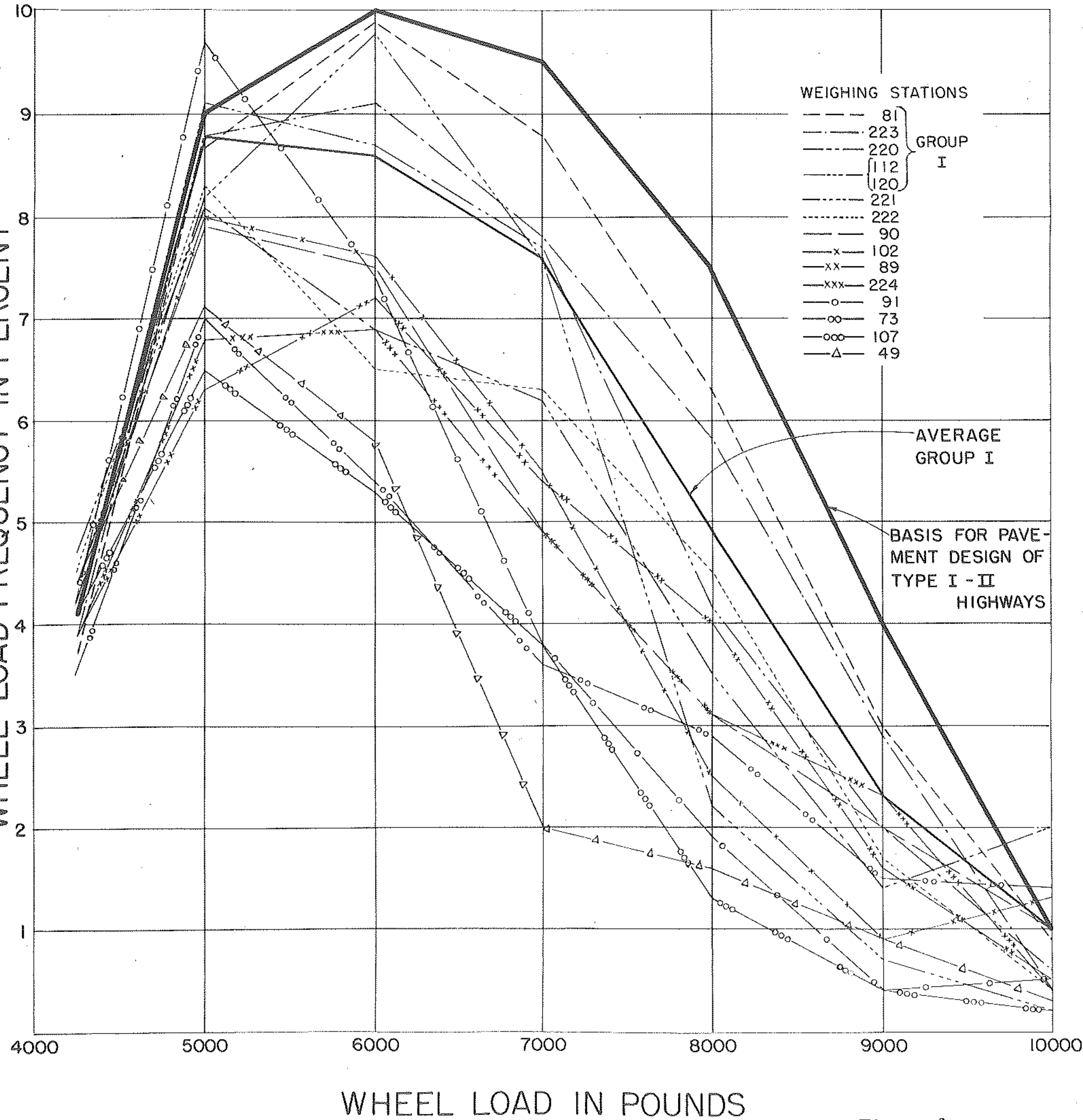

Figure 1

CONCRETE PAVEMENT DESIGN STUDY
TABLE NO. I
DETERMINATION OF WHEEL LOAD FREQUENCY
BASED ON 1936 TRAFFIC COUNT

Weight Station No.	Route No.	Time	Direction of Traffic	Total Vehicles Weighed	Total Axles Weighed	Ratio Axles to Vehicles	WHEEL LOADS																
							Under 4000	%	4000-4400 4250	%	4500-5400 5000	%	5500-6400 6000	%	6500-7400 7000	%	7500-8400 8000	%	8500-9400 9000	%	Over 9400 10,000	%	% Wheel Load
81	US 24		1	2,987	7,638	2.56	4,487	58.7	285	3.7	667	8.7	754	9.9	670	8.8	477	6.3	229	3.0	69	0.9	41.3
223	US 24	Aug.-Sept.	2	7,131	18,235	2.56	11,174	61.3	716	3.9	1,608	8.8	1,657	9.1	1,427	7.8	1,059	5.8	523	2.9	71	0.4	38.7
220	US 16	Feb.-Mar.	2	1,958	4,978	2.54	3,167	63.6	204	4.1	453	9.1	434	8.7	383	7.7	207	4.2	102	2.0	28	0.6	36.4
221	US 12	Apr.-May	2	3,943	9,916	2.51	6,633	66.9	438	4.4	809	8.2	971	9.8	757	7.6	216	2.2	76	0.7	16	0.2	33.1
112 & 120	US 12		1	1,384	3,587	2.59	2,407	67.2	169	4.7	291	8.1	247	6.9	222	6.2	127	3.5	52	1.4	72	2.0	32.8
222	US 112	June-July	2	4,343	10,696	2.46	7,248	67.8	483	4.5	887	8.3	701	6.5	673	6.3	480	4.5	179	1.7	45	0.4	32.2
90	US 24		1	1,127	2,928	2.60	2,023	69.2	130	4.4	231	7.9	221	7.5	144	4.9	91	3.1	60	2.0	28	1.0	30.8
102	US 12		1	1,407	3,192	2.27	2,234	70.0	135	4.2	254	8.0	241	7.6	176	5.5	81	2.5	29	0.9	42	1.3	30.0
89	US 10		1	1,570	3,938	2.51	2,797	71.1	153	3.9	247	6.3	285	7.2	214	5.4	158	4.0	63	1.6	21	0.5	28.9
224	US 10	Sept.-Oct.	2	2,787	6,532	2.34	4,693	71.8	251	3.8	444	6.8	450	6.9	317	4.9	201	3.1	150	2.3	26	0.4	28.2
91	M 78		1	912	2,160	2.37	1,557	72.1	91	4.2	209	9.7	161	7.4	83	3.8	39	1.9	9	0.4	11	0.5	27.9
73	US 16		1	1,244	2,987	2.40	2,228	74.7	103	3.5	212	7.0	160	5.4	107	3.6	90	2.9	46	1.5	41	1.4	25.3
49	US 131		1	856	1,963	2.29	1,527	77.9	87	4.4	140	7.1	113	5.8	40	2.0	32	1.6	18	0.9	6	0.3	22.1
107	US 127		1	1,469	3,197	2.18	2.505	78.4	130	4.1	207	6.5	170	5.3	121	3.8	45	1.3	12	0.4	7	0.2	21.6
				AVERAGE		2.44		69.3		4.1		7.9		7.4		5.6		3.4		1.6		0.7	
				ESTIMATE		2.50	HIGHEST VALUES			4.7		9.7		9.9		8.8		6.3		3.0		2.0	
Estimated Wheel Load Frequency on Basis of Traffic Study										4.0		9.0		10.0		9.5		7.5		4.0		1.0	

CONCRETE PAVEMENT DESIGN STUDY
TABLE II
SUMMARY OF LOADOMETER SURVEY
BY PLANNING AND TRAFFIC DIVISION
1942

Weight Station No.	Route No.	Time	Direction of Traffic	Total Vehicles Weighed	Total Axles Weighed	Ratio Axles to Vehicles	WHEEL LOADS Under 4000	%	4000-4400 4250	%	4500-5400 5000	%	5500-6400 6000	%	6500-7400 7000	%	7500-8400 8000	%	8500-9400 9000	%	Over 9400 10,000	%	% Wheel Load
108	US 16	Aug. 13		117	296	2.53	187	63.2	11	3.7	9	3.0	15	5.1	22	7.4	24	8.1	20	6.8	8	2.7	36.8
109	US 131	Aug. 3		163	416	2.55	266	63.9	18	4.3	35	8.4	21	5.1	30	7.2	29	7.0	13	3.1	4	1.0	36.1
118	US 112 US 131	July 30		110	306	2.78	200	65.3	10	3.3	18	5.9	27	8.8	25	8.2	21	6.9	4	1.3	1	0.3	34.7
89	US 10 M 15	Aug. 7		183	457	2.49	318	69.6	16	3.5	25	5.5	38	8.3	28	6.1	24	5.3	6	1.3	2	0.4	30.4
81	US 24 US 25	Aug. 12		195	507	2.60	369	72.8	20	3.9	20	3.9	18	3.6	24	4.7	26	5.1	18	3.6	12	2.4	27.2
112	US 12	July 31		240	592	2.46	450	76.1	16	2.7	28	4.7	30	5.1	31	5.2	31	5.2	6	1.0	0	0	23.9
82	US 25 M 97	Aug. 11		216	479	2.22	366	76.2	15	3.1	30	6.3	18	3.8	18	3.8	19	4.0	7	1.5	6	1.3	23.8
52	US 27	Aug. 6		130	316	2.43	242	76.6	14	4.4	15	4.7	10	3.2	22	7.0	9	2.8	4	1.3	0	0	23.4
27	US 131	Aug. 5		117	290	2.48	227	78.3	8	2.8	9	3.1	10	3.4	13	4.5	15	5.2	7	2.4	1	0.3	21.7
40	US 31	Aug. 4		121	275	2.27	226	82.2	8	2.9	15	5.5	11	4.0	7	2.5	7	2.5	1	0.4	0	0	17.8
				AVERAGE		2.48		72.5		3.5		5.1		5.0		5.7		5.2		2.3		0.8	

CONCRETE PAVEMENT DESIGN STUDY
TABLE III
SUMMARY OF LOADOMETER SURVEY
BY PLANNING AND TRAFFIC DIVISION
1943

Weight Station No.	Route No.	Time	Total Vehicles Weighed	Total Axles Weighed	Ratio Axles to Vehicles	WHEEL LOADS Under 4000	%	4000-4400 4250	%	4500-5400 5000	%	5500-6400 6000	%	6500-7400 7000	%	7500-8400 8000	%	8500-9400 9000	%	Over 9400 10,000	%	% Wheel Load
118	US 112 US 131	July 29	70	197	2.81	103	52.4	6	3.0	25	12.7	27	13.7	18	9.1	14	7.1	4	2.0	0	0.0	47.6
81	US 24 US 25	Aug. 13	201	562	2.79	314	56.0	23	4.1	30	5.3	44	7.8	71	12.6	44	7.8	23	4.1	13	2.3	44.0
112	US 12	July 30	112	315	2.81	179	56.8	15	4.8	26	8.3	25	7.9	38	12.0	22	7.0	6	1.9	4	1.3	43.2
89	US 10 M 15	Aug. 9	104	264	2.54	165	62.4	11	4.2	14	5.3	14	5.3	32	12.1	20	7.6	6	2.3	2	0.8	37.6
109	US 131	Aug. 2	86	225	2.62	148	65.8	10	4.4	22	9.8	11	4.9	12	5.3	13	5.8	6	2.7	3	1.3	34.2
82	US 25 M 97	Aug. 10	130	323	2.48	214	66.3	12	3.7	26	8.0	19	5.9	13	4.0	22	6.8	11	3.4	6	1.9	33.7
108	US 16	Aug. 12	156	406	2.60	269	66.3	20	4.9	22	5.4	30	7.4	25	6.2	19	4.7	13	3.2	8	2.0	33.7
27	US 131	Aug. 5	59	146	2.48	104	71.2	6	4.1	7	4.8	7	4.8	5	3.4	10	6.9	5	3.4	2	1.4	28.8
52	US 27	Aug. 6	64	164	2.56	120	73.2	6	3.7	11	6.7	9	5.5	5	3.0	8	4.9	3	1.8	2	1.2	26.8
40	US 31	Aug. 3	60	136	2.27	104	76.5	4	2.9	10	7.4	7	5.1	5	3.7	3	2.2	3	2.2	0	0.0	23.5
			AVERAGE		2.60		64.7		4.0		7.4		6.8		7.1		6.1		2.7		1.2	

CONCRETE PAVEMENT DESIGN STUDY
TABLE NO. IV
COMPUTATION OF CRITICAL WHEEL LOAD FREQUENCY, PER DAY

Class	Highway Type	Total Daily Traffic Capacity per Lane	Commercial Traffic Percent	Commercial Traffic Per Lane	Estimated Com. Traffic per Lane	Ratio of Axles to Vehicles	Total Daily Axle Loads	Daily Wheel Load Frequency for Each Class						
								4000-4400 4200 4 %	4500-5400 5000 9%	5500-6400 6000 10%	6500-7400 7000 9.5%	7500-8400 8000 7.5%	8500-9400 9000 4%	Over 9400 10,000 1%
I	Express Way Divided Lane	12,000	15	1800	2000	2.6	5200	208	468	520	494	390	208	52
II	Heavy Primary 2 Lane	12,000	8	960	1000	2.6	2600	104	234	260	247	195	104	26
III	Light Primary 2 Lane	12,000	2	240	250	2.6	650	26	59	65	62	49	26	6.5
IV	Secondary 2 Lane	12,000	1	120	125	2.6	325	13	29	33	31	24	13	3

TABLE NO. V
COMPUTATION OF CRITICAL WHEEL LOAD FREQUENCY, PER YEAR

Class	Highway Type	Critical Wheel Load Per Year						
		4200	5000	6000	7000	8000	9000	10,000
I	Express Way - Divided Lane	75,920	170,820	189,800	180,310	142,350	75,920	18,980
II	Heavy Primary - 2 Lane	37,960	85,410	94,900	90,155	71,175	37,960	9,490
III	Light Primary - 2 Lane	9,490	21,353	23,725	22,539	17,794	9,490	2,373
IV	Secondary - 2 Lane	4,745	10,677	11,863	11,270	8,897	4,745	1,187

where:

P = applied load, in pounds

h = thickness of slab, in inches

a = radius of wheel load distribution in inches
The values are given in Table VI

t = radius of relative stiffness, in inches ("t" substituted for "l" in Westergaard's equation) See Formula V

b = radius of resisting section, in inches, and is computed by formula VI

E = Modulus of elasticity of concrete in pounds per square inch

u = Poisson's ratio for concrete

$Z = \frac{t}{L} = 0.2$

Radius of Wheel Load Distribution: Load distribution radii for various wheel loads according to Bradbury (2)* are given below in Table VI.

TABLE VI

Load-Distribution Radius, "a", in inches

Wheel Load P	Location of wheel on slab: Corner	Interior	Transverse Edge	Longitudinal Edge
4000	4.6	4.6	5.5	6.7
5000	5.0	5.2	6.1	7.4
6000	5.3	5.7	6.6	8.0
7000	5.6	6.1	7.0	8.6
8000	5.8	6.5	7.4	9.1
9000	6.0	6.9	7.8	9.6
10000	6.3	7.3	8.1	10.1

*(2) Bradbury, R. D.: "Reinforced Concrete Pavements". Wire Reinforcement Institute, 1938.

<u>The Radius of Relative Stiffness:</u>. In regard to the radius of relative stiffness Bradbury states that, "a concrete pavement slab functions essentially as a flat plate resting upon a continuous but yielding support. Any tendency for the slab to deflect downward when a load is applied on the pavement surface is restrained to a certain extent by an upward induced reaction exerted by the subgrade. The degree of resistance to slab deflection thus offered by the subgrade is dependent upon the stiffness or pressure-deformation properties of the subgrade material. But the tendency of the slab itself to deflect is dependent upon its properties of flexural stiffness. Thus, the net resultant deflection of the slab, which is also the deformation of the subgrade, which, in turn, is a direct measure of the magnitude of reacting subgrade pressure, becomes a function of the relative stiffness of the slab to that of the subgrade. According to Westergaard, this factor termed the 'radius of relative stiffness' - takes the form of a linear dimension and may be expressed by the formula:"

$$t = \sqrt[4]{\frac{Eh^3}{12\ (1-u^2)K}} \quad - - - - - - - - - \text{V}$$

in which:

t = radius of relative stiffness, in inches

E = modulus of elasticity of the concrete, in pounds per square inch

h = slab thickness, in inches

k = subgrade modulus in pounds per cubic inch

u = Poisson's ratio for the concrete

Radius of Resisting Section: The slab thickness obviously has the effect of distributing the bending moment over some effective section of resistance. Also, the radius of load distribution is a factor in determining the extent of moment distribution. According to Westergaard, the equivalent radius of the resisting section may be approximated in terms of the radius of load distribution and slab thickness by the following formulae.

$$b = \sqrt{1.6\, a^2 + h^2} - 6.75h \text{ when "a" is less than } 1.724h \quad - - - - \text{VI}$$

$$b = \text{"a" when "a" is greater than } 1.724h$$

in which

b = equivalent radius of resisting section, in inches

a = radius of wheel load distribution, in inches

h = slab thickness, in inches

Determination of Subgrade Modulus. In Westergaard's original theory it was assumed that the reactions of the subgrade are vertical only and are proportional to the deflections of the slab. The subgrade reaction per unit area at a given point is the product of the deflection at that point and a coefficient of subgrade stiffness "k" which was termed the modulus of subgrade reaction. This modulus is expressed in pounds per square inch per inch of deflection, or pounds per cubic inch.

To make practical use of the analysis one must be able to assign a value to the modulus of subgrade reaction for the particular soil structure with which he is concerned. There are three recognized field procedures by which the load sustaining ability of the subgrade can be measured.

1. Load-displacement tests on rigid plates, in which the loads are applied at the center of rigid plates of relatively small size. The subgrade modulus is obtained from the load-penetration relationship. The method for computing "k" is shown in Figure 2, part 1.

2. Load-deflection tests on flexible circular plates. The load is applied at the center of flexible circular plates of relatively large dimensions. The subgrade modulus "k" is computed by two methods designated as the volumetric displacement method and the deflection method.

Method A. In the volumetric method, the shape of the deflected plate must be determined precisely and its vertical displacement measured in order to estimate accurately the volumetric displacement of the soil that is caused by the test load on the plate. The modulus of subgrade reaction is computed by dividing the load in pounds by the volume of the displaced soil in cubic inches.

Method B. The deflection method of computing "k" offers a much more precise treatment of the problem than any of the methods so far advanced, since it is based upon a rigorous theory for finite circular plates of uniform thickness, symmetrically loaded and supported. The theory of these plates was developed by Ferdinand Schleicher (3). The subgrade modulus can be readily ascertained (from a deflection measured) at any point under the plate from specially prepared graphs. The theory lends itself to the determination of deflections at positions not immediately under the load, a determination which is valuable as a check and means of detecting any distortion of the plate during the test caused by temperature and moisture conditions. See Figure 2, part 2.

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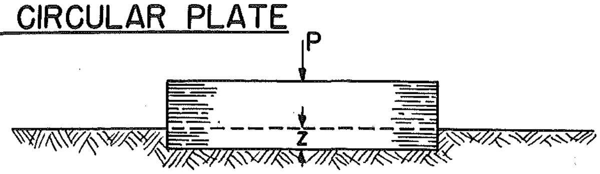
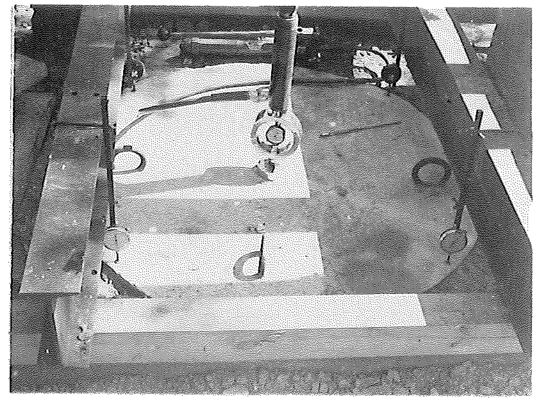
(3) Schleicher, Ferdinand: Kreisplatten auf Elasticher Unterlage. (Circular Plates on Elastic Subgrades) 1926.

1. RIGID CIRCULAR PLATE

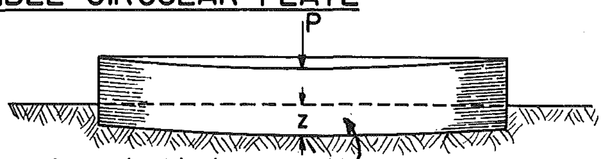


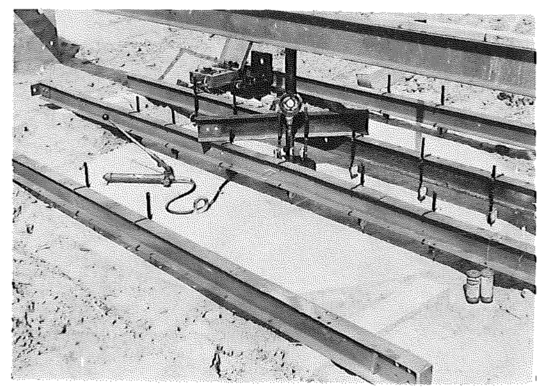


$$k = \frac{P}{A \cdot z}$$

k = subgrade modulus
P = load in pounds
A = area in square inches
z = deflection in inches

2. FLEXIBLE CIRCULAR PLATE





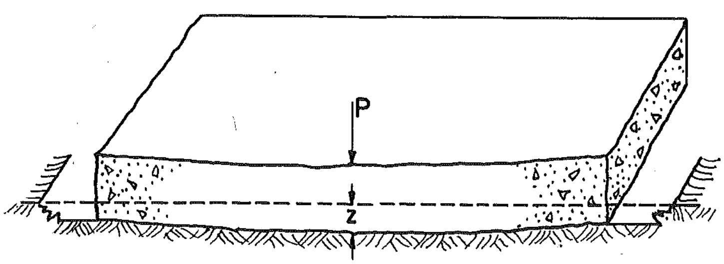
P = load in pounds
V = volumetric displacement of soil in cu. in.
z = deflection in inches
k = subgrade modulus
A = area of plate in sq. inches

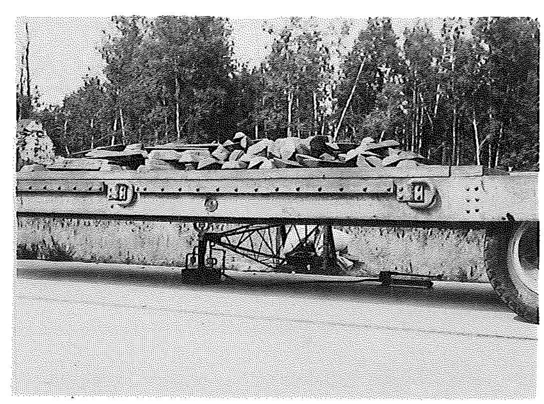
<u>Method A</u>: $k = \frac{P}{V}$

$V = A \cdot z_{av.}$

<u>Method B</u>: "k" can be determined from prepared graphs based on relationships between load deflection and physical properties of the plate.

3. FULL SIZE PAVEMENT SLABS





<u>For Interior</u>: $k = \frac{3(1-u^2)P^2}{16Eh^3z^2}$

<u>For Free Edge</u>: $k = \frac{2(1-u^2)(1+0.4u)^2P^2}{Eh^3z^2}$

k = subgrade modulus, p.c.i.
u = Poissons ratio
E = modulus of elasticity, p.s.i.
P = load in pounds
z = deflection in inches
h = slab thickness in inches

Figure 2

3. Load-deflection tests on full size pavement slabs. This procedure consists of applying test loads at the free edge or interior point of a pavement slab of uniform thickness and of normal size. If the elastic modulus of the concrete in the slab is known, it is possible to estimate the value of the subgrade modulus from the slab deflection under the applied load by means of deflection formulas given by Westergaard (4). See Figure 2, part 3.

All of the three procedures for determining the subgrade modulus have been tried by the Department on various occasions. It has been found that the procedure employing the flexible circular plate, with the subgrade modulus computed by the deflection method (Method 2B), gave much more consistent and reproducible results than did any of the other procedures.

Recent field studies by this method on sandy and sandy clay subgrade materials gave values for subgrade modulus of approximately 200 and 300 p.c.i., respectively. Since it is not anticipated that the subgrade modulus under pavement slabs will exceed 300 p.c.i. under normal construction conditions employing sand subbase material, extreme values for "k" of 100 and 300 p.c.i. have been assumed in the calculations for pavement thickness.

Physical Properties of Concrete. The physical characteristics of concrete involved in Method 2B and in Westergaard's equations for stress determinations are: modulus of elasticity and Poisson's ratio.

The modulus of elasticity of concrete produced under modern design requirements and Michigan specifications is approximately 5,000,000 to 6,000,000 pounds per square inch. A value of 5,000,000 was used in computing unit stresses.

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(4) Westergaard, H. M.: "Computation of Stresses in Concrete Roads." Proceedings of Highway Research Board, 1925. Part I.

Poisson's ratio for concrete was considered to be 0.15 for stress due to load, and zero for warping stresses.

Calculation of Wheel Load Stresses. The maximum unit tensile stress in the concrete has been calculated by equations I, II and III for wheel loads ranging from 4,000 to 10,000 pounds when they occur at the four critical points such as (1) at the interior, (2) at the free longitudinal edge, (3) at a transverse joint edge and (4) at a corner. In the calculation of unit stresses, slab thicknesses from 6 to 12 inches in one-inch increments were considered. Two extreme conditions of subgrade support were assumed, with values for subgrade modulus "k" of 100 and 300 pounds per cubic inch.

The computed stresses for the conditions described above are presented in Table VII. They are also presented graphically in Figures 3,4,5.

Calculations of Temperature - Warping Stresses.

Warping stresses were computed for temperature differentials in the slab of "3h" in spring and summer, "2h" in fall and winter and "h" at night, where "h" equals slab thickness, according to Westergaard's analysis (5)* and assuming Poisson's ratio equal to zero. The equations for computing warping stresses are given as follows:

For interior, longitudinal and transverse joint edges.

$$Swx = S_o \cdot C_x \text{ and } Swy = S_o \cdot C_y \quad \text{- - - - - - - - - - - - - - - - - - VII}$$

$$S_o = \frac{E.e.d.}{2}, \quad C'_x = \frac{12\ L}{t\ \sqrt{8}}, \quad C'_y = \frac{12\ W}{t\ \sqrt{8}}$$

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*(5) Westergaard, H. M.:"Analysis of Stresses in Concrete Pavements Due to Variations of Temperature." Proceedings of Highway Research Board, 1926.

TABLE NO. VII

SUMMARY OF WHEEL LOAD STRESSES DETERMINED BY WESTERGAARD'S EQUATIONS

Wheel Load P	K = 100 - Slab thickness in inches							K = 300 - Slab thickness in inches						
	6"	7"	8"	9"	10"	11"	12"	6"	7"	8"	9"	10"	11"	12"
Wheel Load Stresses at Interior of Slab by Equation No. I														
4000	143	109	87	70	57	48	40	125	96	77	62	51	42	36
5000	169	131	104	84	70	59	50	148	115	92	75	62	52	45
6000	194	151	121	98	81	68	58	169	133	106	87	72	61	52
7000	219	172	137	111	92	78	67	190	149	120	98	82	69	60
8000	242	189	152	124	103	87	75	209	165	134	110	91	78	66
9000	264	206	167	136	114	96	83	226	178	145	120	100	86	73
10000	283	223	180	148	124	105	90	241	192	156	129	109	93	80
Wheel Load Stresses at Longitudinal Edge of Slab by Equation No. II														
4000	198	156	126	103	86	73	63	167	133	108	90	76	64	55
5000	233	184	149	124	104	88	76	195	156	128	107	90	77	66
6000	265	211	172	142	120	102	88	219	177	146	122	104	89	77
7000	293	234	191	159	135	116	99	239	195	162	136	116	99	86
8000	320	258	211	177	150	128	111	259	213	177	150	128	110	96
9000	346	279	230	192	164	140	122	277	229	193	161	139	121	105
10000	369	298	247	207	177	152	132	292	243	204	173	151	129	113
Wheel Load Stresses at Transverse Edge of Slab by Equation No. II														
4000	221	171	137	112	93	77	66	190	149	120	98	82	69	59
5000	262	205	165	134	112	94	80	223	177	143	117	99	84	71
6000	301	236	190	156	130	109	94	254	202	164	136	114	97	83
7000	337	266	215	176	148	125	108	284	227	185	153	130	110	95
8000	373	294	238	198	166	140	120	311	250	204	170	144	123	106
9000	404	322	260	216	182	154	132	335	272	222	185	158	135	117
10000	437	347	285	235	199	168	145	360	291	242	202	171	147	127
Wheel Load Stresses at Corner of Slab by Equation No. III														
4000	202	155	123	100	83	70	59	179	139	111	91	76	64	55
5000	245	188	149	122	101	85	73	214	167	135	110	92	78	67
6000	287	221	176	143	119	101	86	249	195	157	129	108	92	79
7000	325	252	201	164	137	116	99	280	221	179	147	123	105	90
8000	366	284	226	185	154	130	112	313	247	200	165	139	118	102
9000	405	315	252	205	171	145	125	344	273	221	183	154	131	113
10000	439	341	274	224	187	158	136	368	294	239	199	167	143	123
Wheel Load Stresses at Corner of Slab by Equation No. IV														
4000	282	212	166	132	108	90	76	262	199	157	126	104	87	74
5000	346	261	203	163	134	112	95	318	243	192	155	128	107	91
6000	409	309	241	194	159	133	113	373	287	226	183	151	127	108
7000	469	356	278	224	184	154	131	426	328	260	211	174	146	125
8000	531	403	316	254	210	175	149	479	369	294	239	197	166	142
9000	591	450	353	284	234	196	167	530	411	326	266	220	185	158
10000	646	493	388	313	258	216	183	574	447	356	291	241	203	174

WHEEL LOAD STRESSES

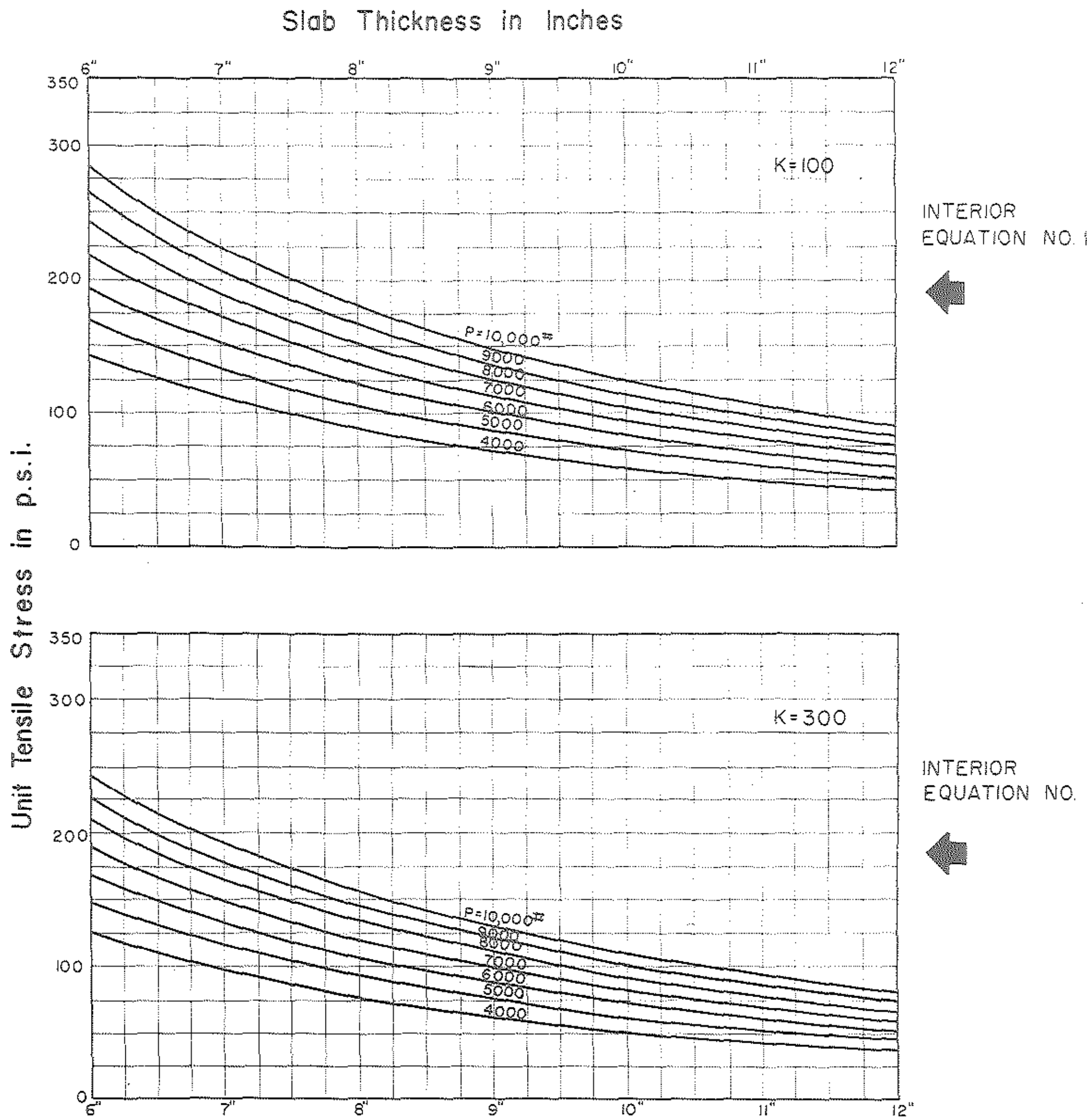

Figure 3

WHEEL LOAD STRESSES

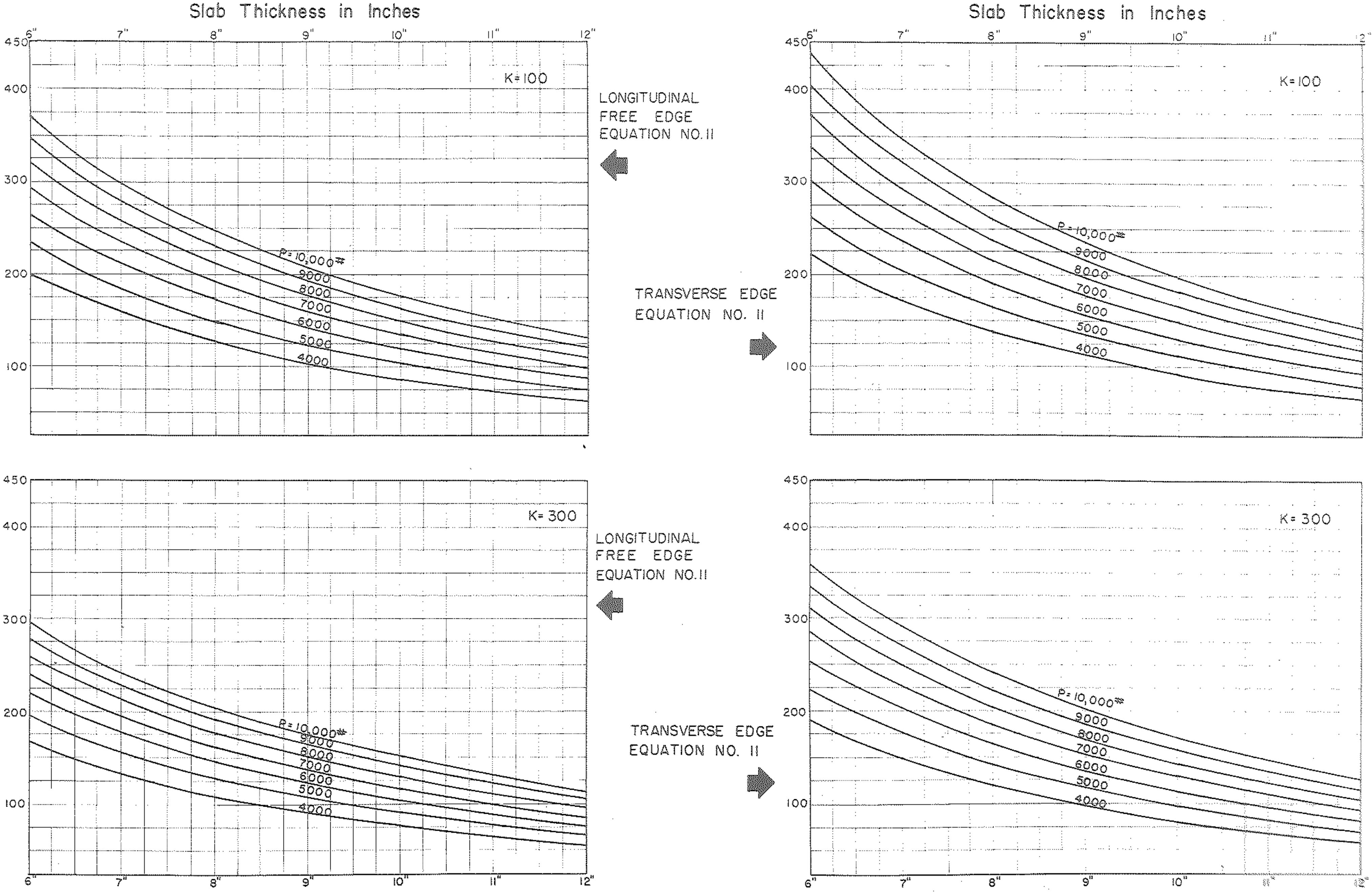

Figure 4

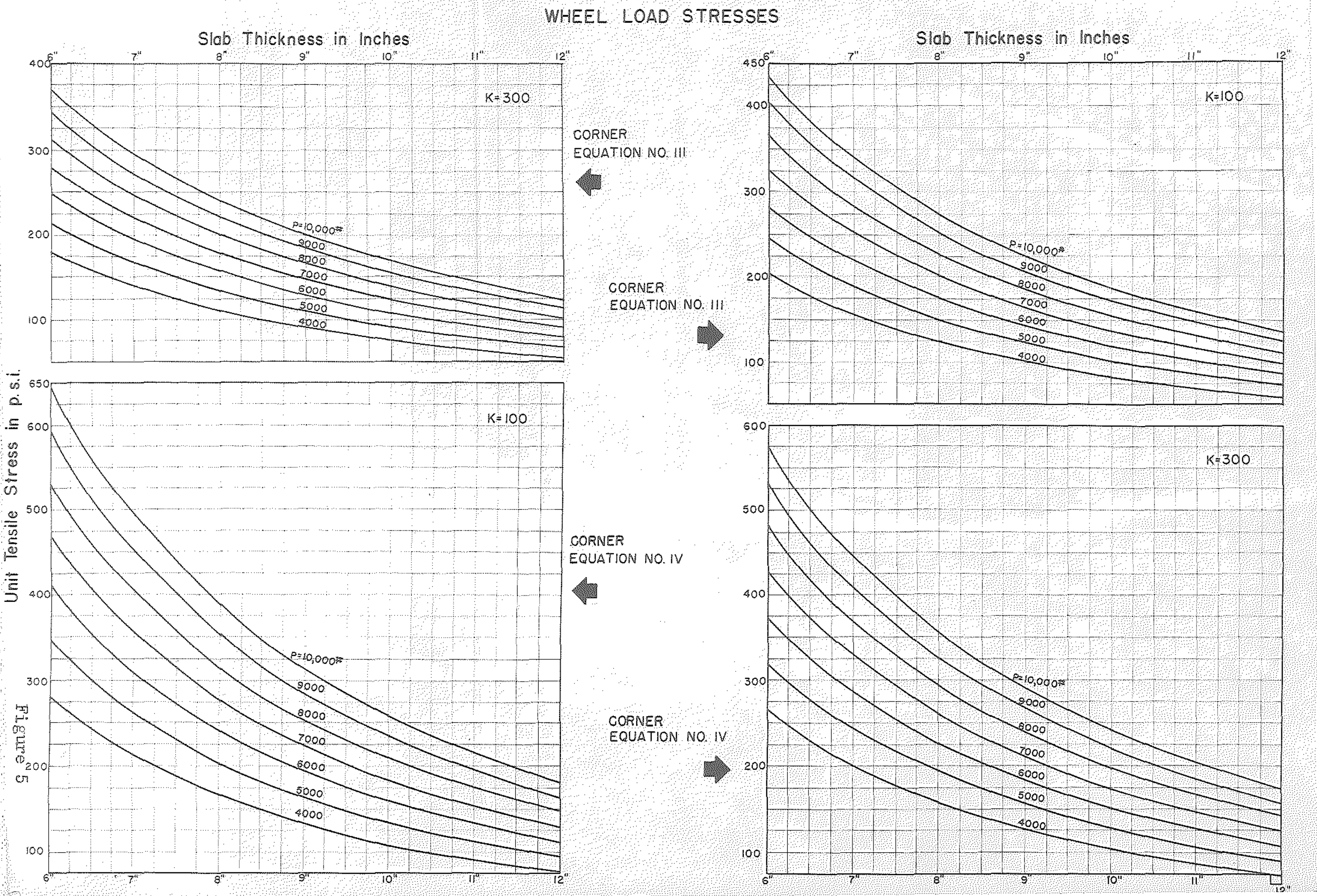
WHEEL LOAD STRESSES
Slab Thickness in Inches
Unit Tensile Stress in p.s.i.
K=300
K=100
CORNER
EQUATION NO. III
CORNER
EQUATION NO. IV
P=10,000#
9000
8000
7000
6000
5000
4000

Figure 5

Near corner of slab, according to Bradbury (6)*

$$Sw_c = \frac{E.e.d.}{3\ (1-u)} \sqrt{\frac{a}{t}} \quad \text{which reduces to}$$

$$Sw_c = 9.167.d \sqrt{\frac{a}{t}} \qquad \text{- VIII}$$

where:

Sw_x and Sw_y = Warping stress in pounds per square inch in x (transverse) and y (longitudinal) direction respectively

Sw_c = Warping stress near corner of slab

Cx and Cy = Coefficients defined by graph in figure 6 in relationship with C'_x and C'_y

E = Modulus of elasticity of concrete in p.s.i.

e = Thermal coefficient of concrete

d = Temperature differential between top and bottom of slab

t = Radius of relative stiffness in inches (Equation V)

L = Length of slab between joints in feet

W = Width of slab in feet

For slab lengths greater than (12t) the warping stresses at a distance of $\pi\ t\ \sqrt{2}$ from slab end was considered to be 1.043 . S_c, in accordance with Bradbury (6)*. A complete summary of warping stresses for critical points is presented in Table VIII and IX.

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(6)* Bradbury, R. D.: Reinforced Concrete Pavements. Wire Reinforcement Institute, 1938.

Subgrade Friction Stresses

The subgrade friction stresses in the concrete at the middle of the slabs "L" feet long were determined by the following equation:

$$S_f = \frac{f.W.L.}{288} \quad \text{-----------------------} \quad IX$$

where:

S_f = Friction stress in pounds per square inch

f = Coefficient of subgrade friction

W = Unit weight of concrete assumed at 150 pounds per cubic foot

L = Length of slab in feet

Laboratory and field studies by the Department and other Highway Organizations indicate that the subgrade friction coefficient varies between approximately 1.0 for light granular soils to 2.0 for heavier clay soils. A friction coefficient of 1.0 was used with "k" equal to 100 and a value of 2.0 with "k" equal to 300 for the determination of slab thickness. A value of 1.5 was assumed for "k" = 200. The calculated friction stresses are presented in table X

TABLE X

SUMMARY OF SUBGRADE FRICTION STRESSES

Length of Slab	Stress p.s.i. k = 100 f = 1.0	Stress p.s.i. k = 300 f = 2.0
10	5.2	10.4
15	7.8	15.6
20	10.4	20.8
30	15.6	31.2
40	20.8	41.6
60	31.2	62.4
100	52.0	104.0
140	72.8	145.6
200	104.0	208.0

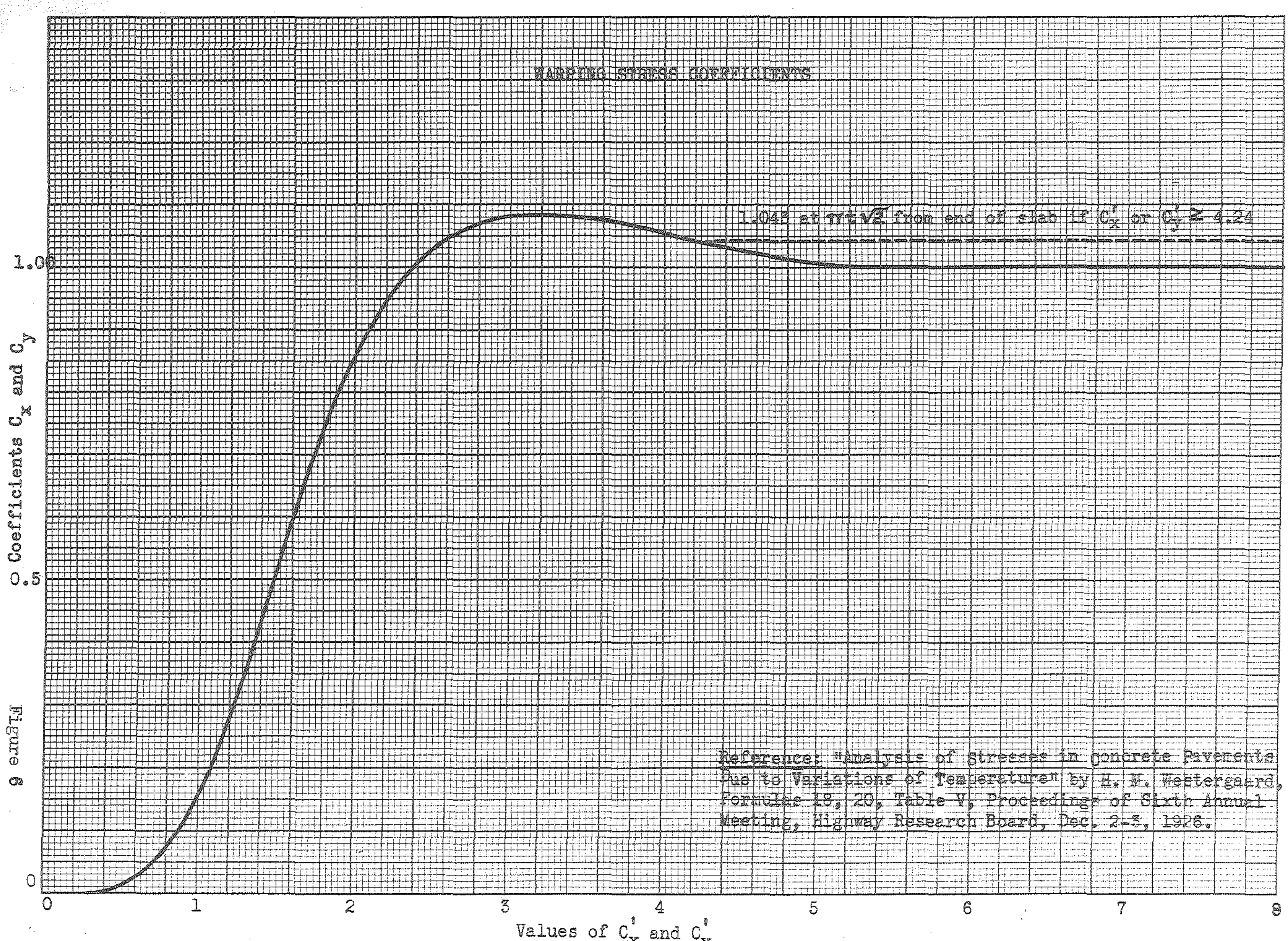

WARPING STRESS COEFFICIENTS
1.043 at $\pi \ell \sqrt{2}$ from end of slab if C_x' or $C_y' \geq 4.24$
1.00
0.5
0
Coefficients C_x and C_y
0
1
2
3
4
5
6
7
8
Values of C_x' and C_y'
Reference: "Analysis of Stresses in Concrete Pavements Due to Variations of Temperature" by H. M. Westergaard, Formulas 18, 20, Table V, Proceedings of Sixth Annual Meeting, Highway Research Board, Dec. 2-3, 1926.

Figure 9

TABLE NO. VIII

SUMMARY OF WARPING STRESSES

AT INTERIOR AND LONGITUDINAL EDGE OF SLAB BY EQUATION VII

AT A POINT MIDWAY BETWEEN ENDS

Slab Thickness In Inches	LENGTH OF SLAB IN FEET 10		11		15		20		30		40		60		at $\frac{16t^*}{12}$		At a distance of $\frac{\pi t\sqrt{2}}{12}$ feet from end of slab	
	k= 100	k= 300	k= 100	k= 300	k= 100	k= 300	k= 100	k= 300	k= 100	k= 300	k= 100	k= 300	k= 100	k= 300	k= 100	k= 300	k= 100	k= 300
6	103	184	129	211	220	263	265	267	258	248	257	248	248	248	248	248	258	258
7	87	173	113	208	221	297	295	315	311	294	300	289	289	289	289	289	302	302
8	69	159	96	195	214	311	314	350	358	340	343	330	330	330	330	330	344	344
9	60	141	81	178	197	319	323	394	403	391	390	372	371	371	371	371	388	388
10	48	124	68	163	181	320	326	425	443	444	442	416	413	413	413	413	430	430
11	41	109	59	146	163	313	319	443	476	493	491	458	455	454	454	454	473	473
12	35	94	50	130	147	300	307	457	517	538	539	500	500	495	495	495	516	516

TABLE IX

WARPING STRESSES AT CORNER OF SLAB BY EQUATION VIII

Slab Thickness In Inches	LOAD ON SLAB IN POUNDS 4000		5000		6000		7000		8000		9000		10000	
	k= 100	k= 300	k= 100	k= 300	k= 100	k= 300	k= 100	k= 300	k= 100	k= 300	k= 100	k= 300	k= 100	k= 300
6	21	24	22	25	23	26	23	27	24	27	24	28	25	29
7	23	27	24	28	25	29	26	30	26	30	27	31	27	31
8	25	29	27	30	27	31	28	32	29	33	29	33	30	34
9	27	31	28	33	29	34	30	35	31	35	31	36	32	37
10	29	34	30	35	31	36	32	37	33	38	33	38	34	40
11	31	36	32	37	33	38	34	39	35	40	35	41	36	42
12	33	38	34	39	35	41	36	42	37	42	37	43	38	44

*t = radius of relative stiffness, See Equation No. V, page 12.

Determination of Combined Unit Stresses

Unit stresses including those produced by wheel loads, warping and friction forces have been combined for each of the critical load positions designated as the interior, longitudinal free edge, transverse joint edge, and corner. In the case of the transverse edge and corner positions, combined stresses have also been computed considering 50% load transfer without impact and no load transfer with 1.50 impact.

Special combined stress summaries were prepared for the interior and longitudinal free edge of continuous slab construction at a distance of $\pi t \sqrt{2}$ from the free ends when the slab lengths in feet were longer than 12t, where "t" equals radius at relative stiffness (See Equation V).

The combined stresses were calculated separately for warping and friction stresses and for warping, friction and load stresses as follows:

1. Warping and friction stresses at interior and longitudinal free edge

   $Ssd = S_{w3} - S_f$ for summer days
   $Ssn = -S_{w1} - S_f$ for summer nights
   $Swd = S_{w2} + S_f$ for winter days
   $Swn = -S_{w1} + S_f$ for winter nights

2. Warping stresses for transverse edge

   $Ssd = S_{w3}$ for summer days
   $Ssn = S_{w1}$ for summer nights
   $Swd = S_{w2}$ for winter days
   $Swn = S_{w1}$ for winter nights

3. Warping and friction stresses at corner

   $Ssd = S_{w3} - S_f$ for summer days
   $Ssn = S_{w1} - S_f$ for summer nights
   $Swd = -S_{w2} + S_f$ for winter days
   $Swn = S_{w1} + S_f$ for winter nights

4. Combined warping, friction and load stresses

$S_c = Ssd + S_L$ for summer days

$S_c = Ssn + S_L$ for summer nights

$S_c = Swd + S_L$ for winter days

$S_c = Swn + S_L$ for winter nights

where S represents unit stress and subscripts denote:

s, Summer	f, Friction
n, Night	L, Load
d, Day	c, combined unit stress
w, Warping	1-2-3,=1h-2h-3h or temperature differential where h = slab thickness

Free body diagrams illustrating the action of the combined unit stresses for various load positions and time of day and year are presented in figure 7.

Method of Combining Wheel-Load Applications With Warping. It is obvious that critical combinations of load and warping stresses at any given section of slab can occur only for a few hours per day during certain seasons of the year. Therefore, in order to arrive at a logical estimate as to the number of wheel load applications which can be combined with temperature-warping, the following assumptions have been made:

1. Approximately 75 percent of commercial traffic occurs between 6 A. M. and 6 P. M. (considered daytime traffic). This figure is derived from Table XI, which is compiled from traffic studies by the Planning and Traffic Division.
2. Critical warping takes place during a 5-hour period in the daytime and during the same period of time in the night. This is established by pavement temperature studies by the Public Roads Administration and verified by observations on the Michigan Test Road.

3. The critical period of stress application may be reduced one-half because of gradual variation of temperature differential in the slab for the 5-hour period.

4. In Michigan, 25 percent of days in the year are bright according to U. S. Weather Bureau records at East Lansing. At night, 100 percent of wheel loads are considered critical because of the considerable temperature changes during that period which will influence warping of the slab more than temperature fluctuations occurring on dull days.

5. The numerically greatest value of these daily maxima is apt to occur either during the spring and summer months, or fall and winter months. Each period is considered 1/2 year.

On the basis of the above assumptions, the percent of total critical wheel loads is computed as shown below, where each factor is taken in order of presentation above.

Day Time

For spring and summer: $0.75 \times \frac{5}{12} \times 1/2 \times 0.25 \times 1/2 \times 100 = 2.0\%$

For fall and winter : $0.75 \times \frac{5}{12} \times 1/2 \times 0.25 \times 1/2 \times 100 = 2.0\%$

Night Time

For spring and summer: $0.25 \times \frac{5}{12} \times 1/2 \times 1 \times 1/2 \times 100 = 2.6\%$

For fall and winter : $0.25 \times \frac{5}{12} \times 1/2 \times 1 \times 1/2 \times 100 = 2.6\%$

On the basis of the above assumptions and calculations, a value of 4% has been assumed for the purpose of computing the number of wheel load applications per year combined with warping. The total critical wheel load applications per year considered in the computation of slab thickness are given in Table XII.

TABLE XI

HOURLY DISTRIBUTION OF ANNUAL AVERAGE DAILY TRUCK TRAFFIC
FROM A COMPOSITE OF 3 TRAFFIC COUNT STATIONS

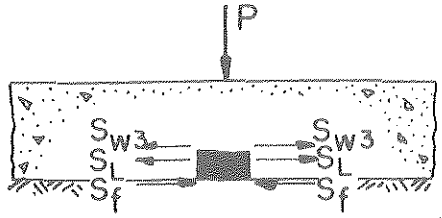
Based on 1 Week from Each Month of the Year 1936

By Planning and Traffic Division
Michigan State Highway Department

	SINGLE TRUCKS		TRAILER COMBINATIONS		TOTAL TRUCKS & TRAILER COMBINATIONS	
HOUR	VOLUME	PERCENT	VOLUME	PERCENT	VOLUME	PERCENT
12 PM - 1 AM	16.79	1.36	53.85	5.22	70.64	3.12
1 AM - 2 AM	14.75	1.20	57.06	5.53	71.81	3.17
2 AM - 3 AM	15.85	1.28	52.72	5.11	68.57	3.03
3 AM - 4 AM	17.66	1.43	51.78	5.01	69.44	3.06
4 AM - 5 AM	18.99	1.54	49.59	4.80	68.58	3.03
5 AM - 6 AM	26.16	2.12	50.07	4.85	76.23	3.36
6 AM - 7 AM	38.32	3.11	47.18	4.57	85.50	3.77
7 AM - 8 AM	51.66	4.19	36.27	3.51	87.93	3.88
8 AM - 9 AM	69.45	5.63	32.18	3.12	101.63	4.48
9 AM - 10 AM	84.97	6.89	36.31	3.52	121.28	5.35
10 AM - 11 AM	86.73	7.03	37.69	3.65	124.42	5.49
11 AM - 12 M	86.66	7.02	38.68	3.75	125.34	5.53
12 M - 1 PM	78.87	6.39	40.37	3.91	119.24	5.26
1 PM - 2 PM	82.88	6.72	41.08	3.98	123.96	5.47
2 PM - 3 PM	84.99	6.89	41.70	4.04	126.69	5.59
3 PM - 4 PM	83.39	6.76	41.81	4.05	125.20	5.53
4 PM - 5 PM	83.95	6.81	43.17	4.18	127.12	5.61
5 PM - 6 PM	78.91	6.40	39.10	3.79	118.01	5.21
6 PM - 7 PM	57.81	4.69	37.46	3.63	95.27	4.20
7 PM - 8 PM	47.12	3.82	37.31	3.61	84.43	3.73
8 PM - 9 PM	36.11	2.93	39.29	3.80	75.40	3.33
9 PM - 10 PM	27.74	2.25	38.60	3.74	66.34	2.93
10 PM - 11 PM	23.13	1.87	42.59	4.12	65.72	2.90
11 PM - 12 PM	20.64	1.67	46.59	4.51	67.23	2.97
24-Hour Total	1233.53	100.00	1032.45	100.00	2265.98	100.00
6 AM - 6 PM Total		73.84		46.07		61.17

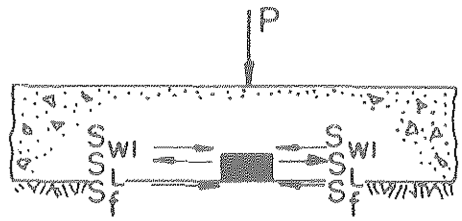
COMBINED STRESSES FOR INTERIOR & LONGITUDINAL FREE EDGE

SUMMER



DAYTIME

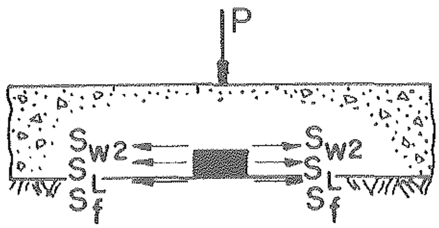
$S_c = S_L + S_{sd} = S_L + S_{w3} - S_f$



NIGHT TIME

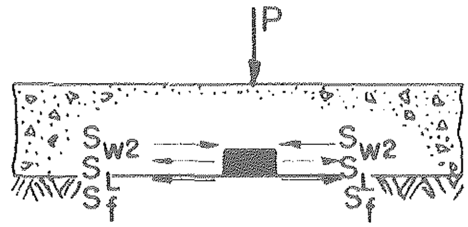
$S_c = S_L + S_{sn} = S_L - S_{wI} - S_f$

WINTER



DAYTIME

$S_c = S_L + S_{wd} = S_L + S_{w2} + S_f$

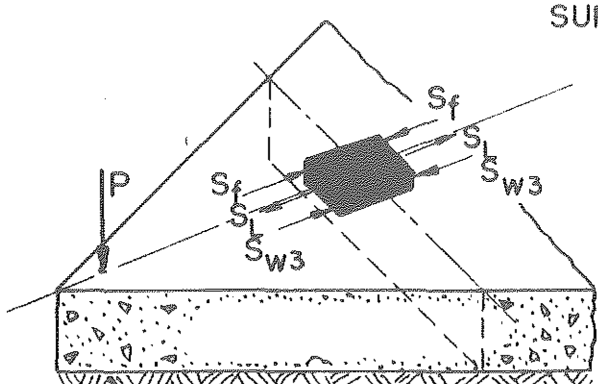


NIGHT TIME

$S_c = S_L + S_{wn} = S_L - S_{wI} + S_f$

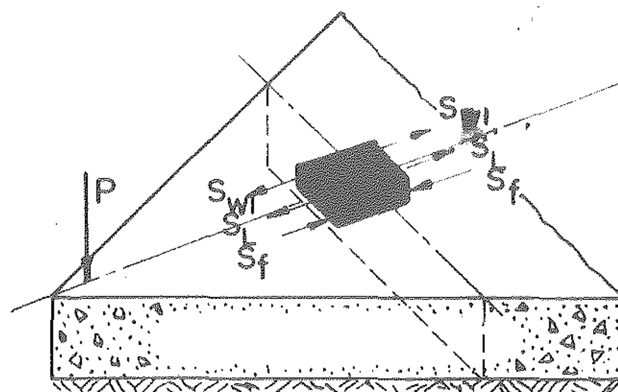
COMBINED STRESSES AT CORNER

SUMMER



DAY TIME

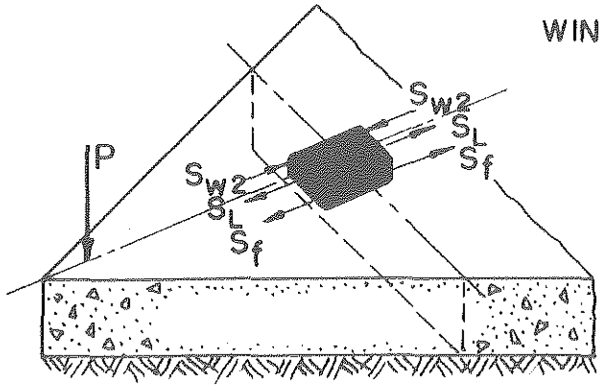
$S_c = S_L + S_{sd} = S_L + S_{w3} - S_f$



NIGHT TIME

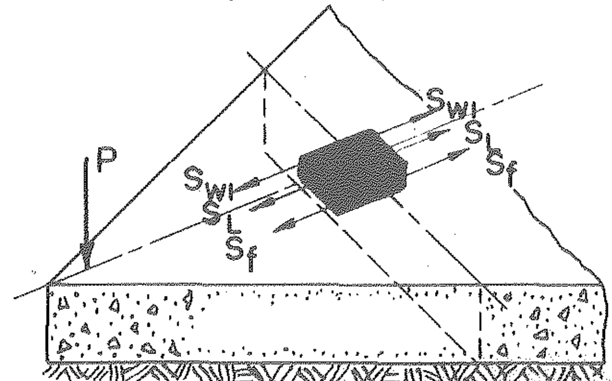
$S_c = S_L + S_{sn} = S_L + S_{wI} - S_f$

WINTER



DAYTIME

$S_c = S_L + S_{wd} = S_L - S_{w2} + S_f$



NIGHT TIME

$S_c = S_L + S_{wn} = S_L + S_{wI} + S_f$

Figure 7

TABLE XII

NUMBER OF CRITICAL WHEEL LOAD APPLICATIONS PER YEAR

	HIGHWAY CLASS I		HIGHWAY CLASS II	
Wheel Loads	Total Critical Wheel Loads	Number Combined with warping 4% Spring & Summer or 4% Fall & Winter	Total Critical Wheel Loads	Number Combined with warping 4% Spring & Summer or 4% Fall & Winter
4000	75,920	3,037	37,960	1.518
5000	170,820	6,833	85,410	3,416
6000	189,800	7,592	94,900	3,796
7000	180,310	7,212	90,155	3,606
8000	142,350	5,694	71,175	2,847
9000	75,920	3,037	37,960	1,518
10000	18,980	759	9,490	380
	HIGHWAY CLASS III		HIGHWAY CLASS IV	
4000	9,490	380	4,745	190
5000	21,353	854	10,677	427
6000	23,725	949	11,863	475
7000	22,539	902	11,270	451
8000	17,794	712	8,897	356
9000	9,490	380	4,745	190
10000	2,373	95	1,187	48

DETERMINATION OF SLAB THICKNESS

The determination of slab thickness is based upon the crack expectancy of the pavement slab. The "crack expectancy" of concrete pavements is defined as the number of years elapsing between construction of the pavement slab and the time at which cracks are expected to occur. It being assumed that the cracks when they occur are caused by fatigue in the concrete due to repetitions of unit stresses of magnitudes exceeding 50 percent of the ultimate rupture strength of the concrete. Laboratory studies on the fatigue of concrete indicate that unit stresses of magnitudes less than 50 percent of the ultimate rupture strength of the concrete are not harmful to the structural integrity of the concrete structure.

Stress Cycles. When computing crack expectancy for any part of a pavement slab it is necessary to associate maximum stress possibilities with stress cycles and the principle of fatigue. A stress cycle is considered as one complete change in unit stress from minimum to maximum. The number of stress cycles per year which are caused by the frequency of load applications and temperature changes during that period are determined in the following manner:

1. Stress cycles due to load and temperature; cycles varying from:

$$S_{sd} \text{ to } (S_{sd} + S_L)$$
$$S_{wd} \text{ to } (S_{wd} + S_L)$$
$$S_{sn} \text{ to } (S_{sn} + S_L)$$
$$S_{wn} \text{ to } (S_{wn} + S_L)$$

with load frequencies as given in Table V.

2. Stress cycles due to warping only:

$$S_{sd} \text{ to } S_{sn}$$
$$S_{wd} \text{ to } S_{wn}$$

Since 1/2 year is considered as spring and summer or fall and winter and 25 percent of the days in the year are bright, the number of complete warping cycles per year are computed as follows:

$$1/2 \times 0.25 \times 365 = 46 \text{ cycles}$$

$$1/2 \times 0.25 \times 365 = 46 \text{ cycles}$$

3. Stress cycles where friction force varies from:

$+ S_f$ to $-S_f$ with an annual frequency of unity

4. Any other possible stress cycles were assumed not to affect the crack expectancy.

Fatigue of Concrete. The number of stress applications to cause failure of the concrete was determined from the specially prepared graph presented in Figure 8. This tentative fatigue graph has been prepared on the basis of concrete fatigue data determined by the Illinois Division of Highways (7)*, which has been interpreted graphically by Bradbury. The graph presented by Bradbury has been modified and extended voluntarily by the authors to take into consideration the relationship of fatigue due to stress repetitions when the loads are applied to concrete slabs already under varying magnitudes of stress. This action is based upon the work of German investigators who have made similar studies with metals (8)*. In Bradbury's interpretation of the fatigue principle he assumes that the concrete in the pavement has zero stress at time of load application.

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*(7) Illinois Division of Highways, Engineering Report No. 34-1, Fatigue Curves according to R. D. Bradbury. Reinforced Concrete Pavements-1938.

*(8) Report No. 42, October 21, 1933. Verein Deutscher Ingenieuve (VDI)

Calculation of Crack Expectancy. The design factors which affect crack expectancy in pavement slabs are: thickness, slab length, subgrade modulus, uniformity of cross section, and traffic volume. The crack expectancy for each case is calculated separately in the following manner:

With the subgrade assumed constant, slab length and thickness fixed, the warping stresses and load stresses are computed for critical points in the slab. These stress values are then combined and tabulated. The maximum and minimum stresses for each wheel load class are selected for the spring and summer seasons, and also for the fall and winter. Repetitions of cycles to failure are read from Figure 8. The repetitions of cycles to failure are designated "R". For each wheel load class the annual load frequency is selected from Table XII. This value is termed "r". The ratio $\frac{r}{R}$ is then computed. A similar computation is made for warping stresses only, again using Figure 8 for R, but using r =46 (see stress cycles). The contribution of stress due to friction is negligible, since the annual frequency is unity. The ratios $\frac{r}{R}$ are summed, and the reciprocal of this value is the crack expectancy in years for the particular condition considered. The procedure employed in calculating crack expectancy is shown in the following example.

EXAMPLE OF CRACK EXPECTANCY CALCULATIONS

Assume for example a point at the interior of a slab, having a length of 20 feet, a thickness of 6", k = 100, and f = 1.0. The crack expectancy computation is as follows:

From previous calculations the following warping and warping plus friction stresses were obtained:

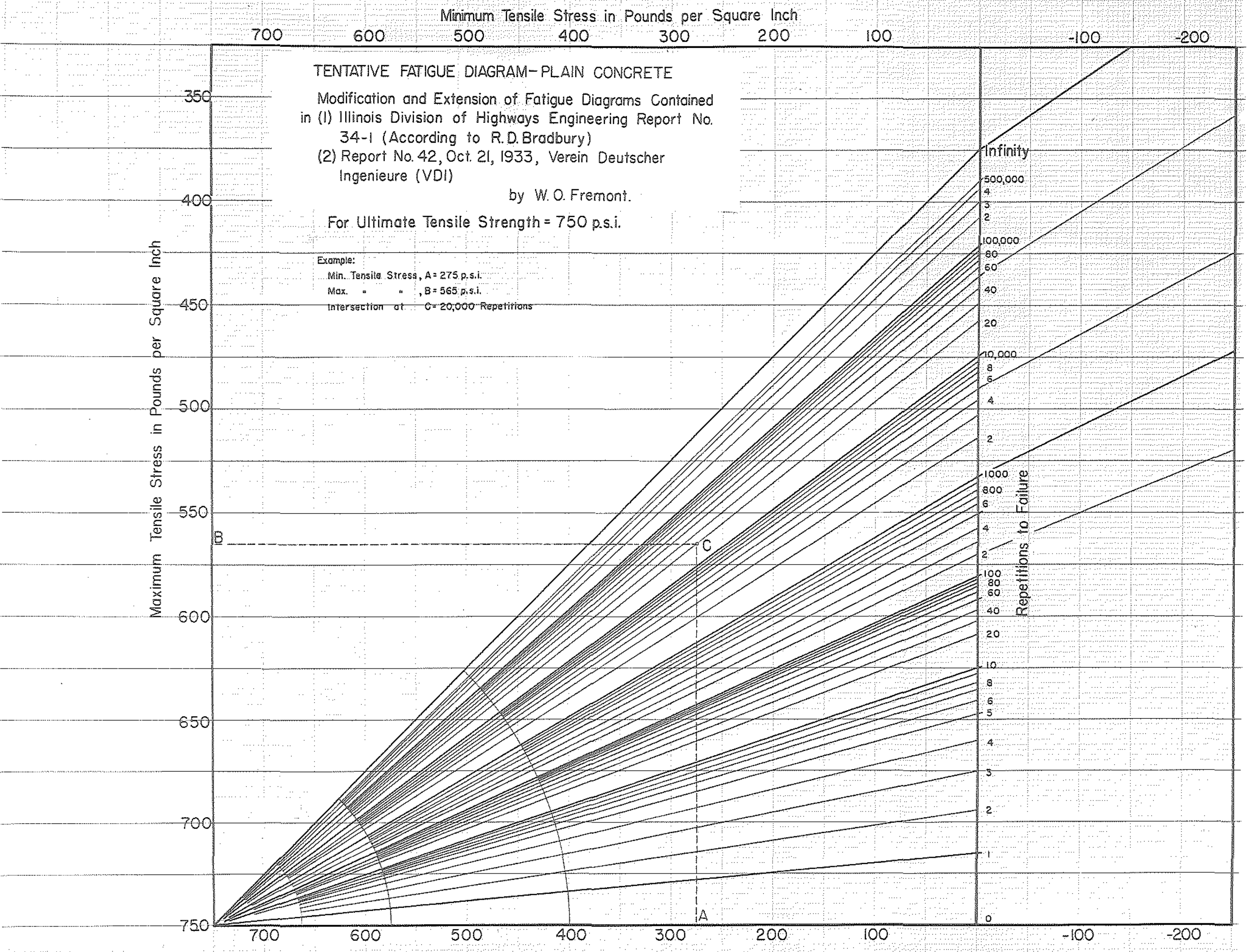

Minimum Tensile Stress in Pounds per Square Inch
TENTATIVE FATIGUE DIAGRAM– PLAIN CONCRETE
Modification and Extension of Fatigue Diagrams Contained in (1) Illinois Division of Highways Engineering Report No. 34-1 (According to R. D. Bradbury)
(2) Report No. 42, Oct. 21, 1933, Verein Deutscher Ingenieure (VDI)
by W. O. Fremont.
For Ultimate Tensile Strength = 750 p.s.i.
Example:
Min. Tensile Stress, A = 275 p.s.i.
Max. " " , B = 565 p.s.i.
Intersection at C = 20,000 Repetitions
Maximum Tensile Stress in Pounds per Square Inch
Repetitions to Failure
Infinity
500,000
100,000
10,000
1000
100
10
B
C
A
700
600
500
400
300
200
100
-100
-200
350
450
550
650
750

Figure 8

(A) Warping Stresses from Equation VII

When $d = h$ then $S_{w1} = 88$ p.s.i.

$d = 2h$ then $S_{w2} = 176$ p.s.i.

$d = 3h$ then $S_{w3} = 265$ p.s.i.

(B) Friction Stresses from Equation IX

When $L = 20$ feet then $S_f = 11$ p.s.i.

(C) Combined Warping and Friction Stresses From Figure 4

$S_{sd} = S_{w3} - S_f = 254$ p.s.i.

$S_{sn} = -S_{w1} - S_f = -99$ p.s.i.

$S_{wd} = S_{w2} + S_f = 187$ p.s.i.

$S_{wn} = -S_{w1} + S_f = -77$ p.s.i.

The warping and friction stresses are independent of the load "P" and are combined with load stress S_L as follows:

TABLE XIII

COMBINED LOAD AND WARPING STRESSES

Load P	Load Stresses S_L (from table VII)	Combined Stresses $S_{wd} + S_L$	$S_{wn} + S_L$	$S_{sd} + S_L$	$S_{sn} + S_L$
4000	143	330	66	397	44
5000	169	356	92	423	70
6000	194	381	117	448	95
7000	219	406	142	473	120
8000	242	429	165	496	143
9000	264	451	187	518	165
10000	283	470	206	537	184

To determine crack expectancy, repetitions to failure "R" must be determined from the graph of Figure 8. This necessitates selection of maximum and minimum tensile stresses. These are tabulated as follows:

TABLE XIV

DETERMINATION OF CRACK EXPECTANCY

Load P	Minimum Tensile Stress, from (C)	Maximum Tensile Stress, from Table XIII	R (from fig. 8)	r (from table XII)	$\frac{r}{R}$
	S_{sd}	$S_{sd} + S_L$			
		(Spring and Summer)			
4000	254	397	∞	3037	0
5000	254	423	∞	6833	0
6000	254	448	∞	7592	0
7000	254	473	∞	7212	0
8000	254	496	∞	5694	0
9000	254	518	300,000	3037	0.0101
10000	254	537	70,000	759	0.0108
		(Fall and Winter)			
	S_{wd}	$S_{wd} + S_L$			
4000	187	330	∞	3037	0
5000	187	356	∞	6833	0
6000	187	381	∞	7592	0
7000	187	406	∞	7212	0
8000	187	429	∞	5694	0
9000	187	451	∞	3037	0
10000	187	470	∞	759	0
		(Warping Only)			
	S_{sn}	S_{sd}			
–	-99	254	∞	46	0
	S_{wn}	S_{wd}			
–	-77	187	∞	46	0
	S_f	S_f			
–	-11	+11	∞	1	0
				Total	0.0209

$$\text{Crack Expectancy} = \frac{1}{\frac{r}{R}} = \frac{1}{0.0209} = 48.0 \text{ years}$$

Determination of Slab Thickness For A Desired Pavement Crack Expectancy. The crack expectancy calculation is made for each type of pavement to be considered. These include thickness from 6" to 12", slab lengths from 10 ft. to 200 ft. and various joint spacing. The results are plotted on coordinate paper with pavement thickness against crack expectancy. Curves are drawn joining points of the same slab length. A sample of such a family of curves is shown in Figure 9.

To determine the pavement thickness necessary for a given crack expectancy, it is only necessary to choose a definite time period, on the graph draw a horizontal line through that point and note the abscissae of the points of intersection of this line with the crack expectancy curves. As an illustration see Figure 9. The required time interval is 20 years, and the dashed line at this level intersects the curves as follows:

TABLE XV

PAVEMENT THICKNESS DETERMINATION

Curve No.	Slab Length In Feet	Location	Pavement Thickness In Inches
1.	100	Longitudinal Edge	6.5
2	140	Longitudinal Edge	7.5
3	Any length	Corner	9.3

A design diagram may now be drawn by graphing these data, using slab lengths as abscissae and pavement thicknesses as ordinates. Similar diagrams may be made for each Highway class and for definite design conditions.

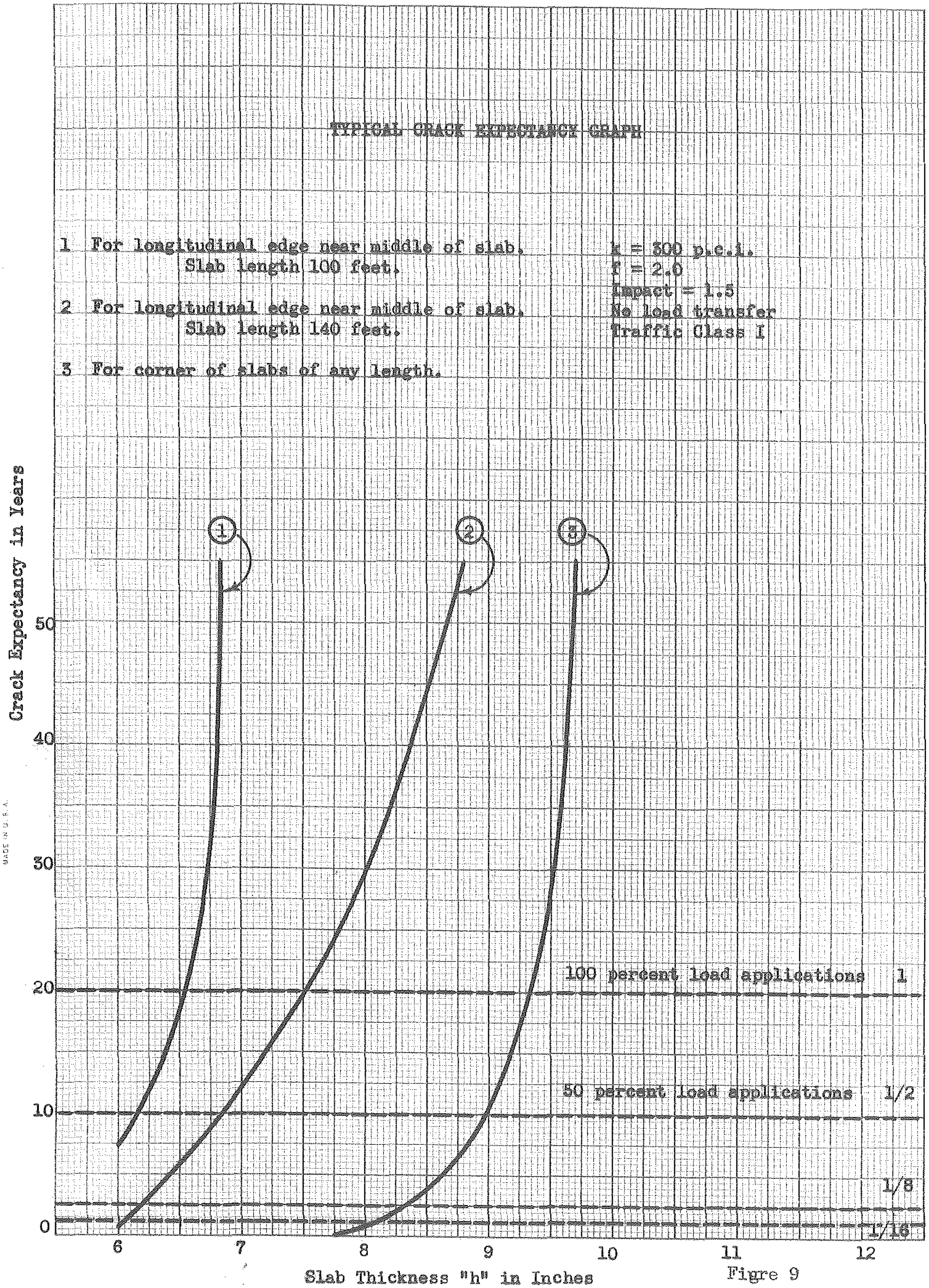
Preparation of Design Diagrams, Uniform Cross Section.

In preparing the design curves for uniform pavements, as shown in Figure 10, the slab thicknesses were determined first for Class I highways. The slab thicknesses for pavements in the other three classes were determined from the design data established from Class I in the following manner:

The load frequencies for traffic Classes II, III, and IV are definite fractions of the frequency for Class I, (See Tables IV and V) and the ratios of these frequencies to that of Class I are respectively 1/2, 1/8, and 1/16. Hence the pavement life period, or crack expectancy, would be expected to be 2, 8, and 16 times that of the expectancy for Class I if the slab thickness remained unchanged.

To determine the pavement thicknesses for a given crack expectancy, a horizontal line is drawn at the given crack expectancy value in a graph similar to that of Figure 9. The abscissae of the points of intersection of this line and each curve gives the thicknesses required for each condition specified for Class I traffic. Since Class II traffic has half the frequency, a horizontal line at one-half the value for Class I traffic will intersect the curves at points whose abscissae give thicknesses required for Class II traffic. Correspondingly, lines at 1/8 and 1/16 the values of Class I crack expectancy yield thicknesses required for Classes III and IV (See Figure 9).

Design curves have also been prepared for two cases of highway loadings, (1) when 100 percent of all wheel loads act at the free edge and corner and (2) when only 50 percent of all wheel loads reach the same positions. For the case of 50 percent wheel load applications the slab



TYPICAL CRACK EXPECTANCY GRAPH
1 For longitudinal edge near middle of slab. Slab length 100 feet.
2 For longitudinal edge near middle of slab. Slab length 140 feet.
3 For corner of slabs of any length.
k = 300 p.c.i.
f = 2.0
Impact = 1.5
No load transfer
Traffic Class I
1
2
3
100 percent load applications 1
50 percent load applications 1/2
1/8
1/16
Crack Expectancy in Years
0
10
20
30
40
50
6
7
8
9
10
11
12
Slab Thickness "h" in Inches

Figre 9

DESIGN DIAGRAMS FOR CONTINUOUS UNIFORM REINFORCED CONCRETE PAVEMENT

FOR HIGHWAY CLASSES I-II-III-IV

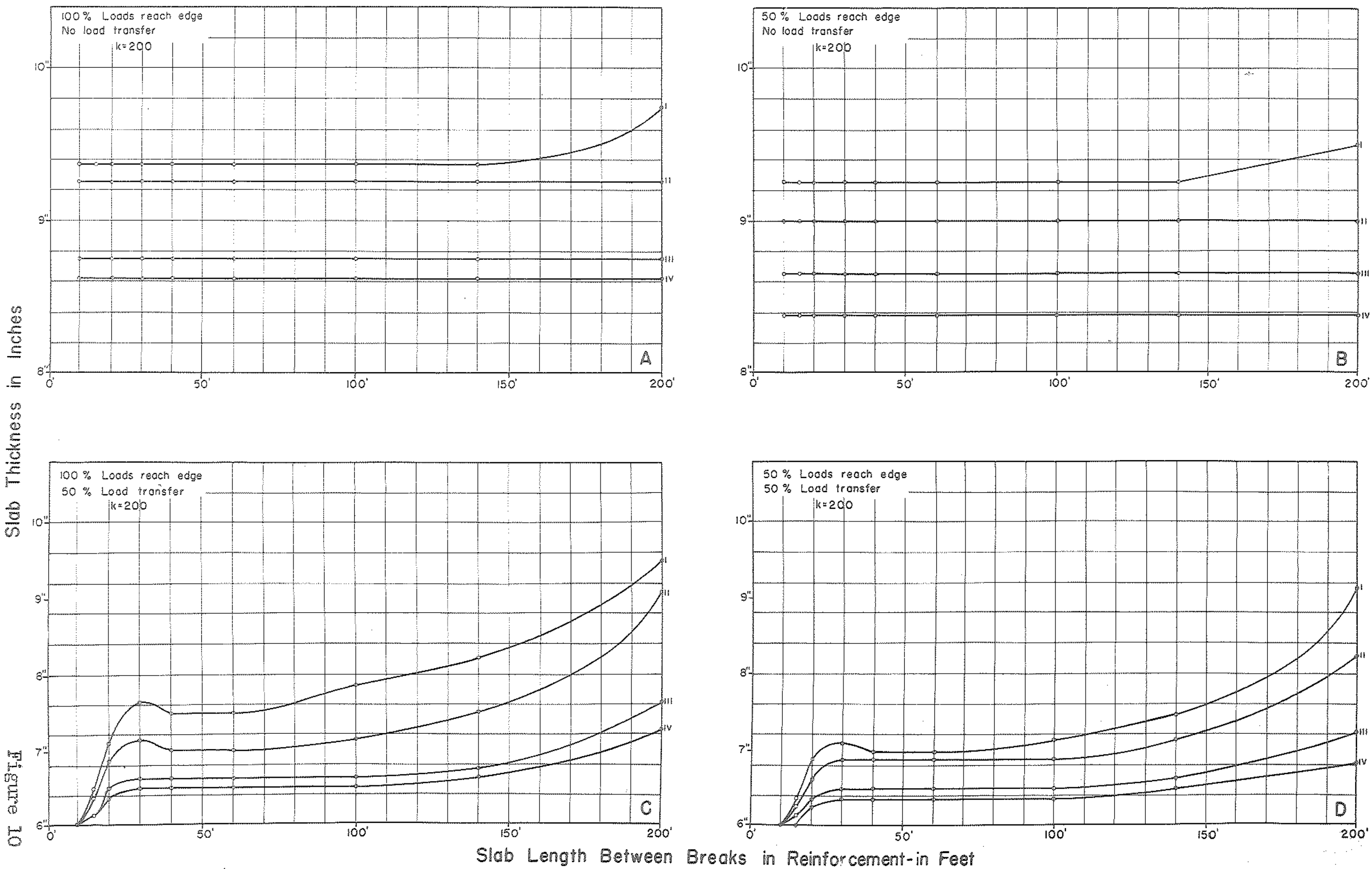


Figure 10

thicknesses for the various design requirements are obtained from the crack expectancy graphs prepared for Class I highways in the manner described above for different highway classes but taking in this case a 10 year period for crack expectancy (See Figure 9).

In constructing the design curves for uniform pavements, given in Figure 10, largest slab thickness was selected for the subgrade modulus and design requirements for "k" = 100, with f = 1.00 and "k" = 300 with f = 2.00. The average of the above found values were used for expressing the slab thickness versus slab lengths for continuous and hinged slabs when "k" = 200 with f = 1.50.

Twelve Foot Lanes or 24 Foot Pavement Widths.

As previously mentioned, all design calculations have been based upon 11 foot lanes or 22 foot pavements. However, some 24 foot pavements have been constructed and others are contemplated. In that connection, supplementary tables and graphs were prepared showing the effect on slab thickness of increqsing "W" (width) by one foot.

The data from the study indicate that critical stresses in pavements of uniform thickness occur at the corners, and since corner stresses are not affected by changes in slab width it is obvious that the increase in width does not affect the slab thickness at least for a change in lane width from 11 to 12 feet. Therefore, the design data previously presented for the 22 foot pavement will suffice for 24 foot pavement widths.

With the increase of slab width the steel reinforcement requirements per 100 square feet do not change, but the cross-sectional area of the longitudinal steel must be increased in the ratio of $\frac{24}{22}$. Graphs showing the reinforcement requirements for 22 and 24 foot pavements will be found in Part II under a separate discussion of steel reinforcement in relation to joint spacing.

PART II

JOINT SPACING

The question of joint spacing in concrete pavements has concerned highway engineers for a great many years. At the present time the subject still seems to be quite controversial although there is available sufficient information on slab behavior to afford a logical solution of this problem.

From all viewpoints, an ideal concrete pavement surface would be one consisting of a continuous ribbon of concrete of uniform construction with no breaks in its continuity. Unfortunately such a concrete pavement cannot be achieved under present day practice. Because of its inherent weakness in tension, concrete is highly susceptible to cracking under tensile stresses particularly those induced in pavement slabs by volume changes due to temperature fluctuation. The nearest approach to obtaining an ideal pavement free of transverse cracks is best accomplished by dividing the pavement into sections or slabs, each slab as long as possible consistent with practical design requirements and within economic limitations.

To satisfy these conditions, joints are normally provided in concrete pavements to reduce to safe values the stresses caused by expansion, contraction and warping of the concrete, and by subgrade friction forces. Thus in accordance with the adopted design procedure, presented in Part I, the spacing of joints is limited, although not necessarily fixed, by permissible maximum stress intensities which should provide reasonable freedom from cracks in a period of 20 years. Joints in this catagory are generally known as expansion and plane of weakness, the latter type being commonly designated contraction or dummy joints depending upon the manner in which they are constructed.

Expansion Joint Spacing

It is generally considered in the design of concrete pavements that the spacing of expansion joints should be dependent on the allowable compressive stress in the concrete and on the maximum compressive stress created by the expansion of the slab.

It was assumed in the calculations for slab thickness, in Part I, that no forces (tension or compression) are acting at the ends of slabs where the reinforcement is broken. This requirement may be satisfied by providing an expansion joint at every break in reinforcement.

On the other hand, if the expansion joints are not provided at every break in reinforcement, then additional compressive stresses, not considered in the calculations, will be induced in the pavement during periods when the prevailing air temperature is above that of pouring temperature. These stresses will reduce the tensile stresses on which the calculations are based and, therefore tend to increase the period of crack expectancy which is a desirable feature.

Compressive Strength Versus Compressive Stresses. The average compressive strength of pavement concrete in Michigan, as determined from field specimens at 28 days and from pavement cores at ages of at least one year, is consistently between 4000 and 6000 pounds per square inch. It can be proved mathematically that these unit compressive stress values are in excess of any that might be caused by extreme temperature fluctuations common to Michigan.

For example, assume as an extreme case that a pavement laid during an air temperature of 40°F reached a temperature of 140°F the following summer, which is quite possible in Michigan. If the pavement is fully restrained, the maximum possible unit compressive stress which could occur under a temperature differential of 100°F would be 2500 pounds per square inch as computed by the following equation:

$$S_c = E.e.T \text{ - XI}$$

where:

S_c = Unit compressive stress in pounds per square inch

E = Modulus of elasticity assumed as (5×10^6 p.s.i.)

e = Coefficient of expansion assumed as (.000005)

T = Temperature differential (100°F)

At plane of weakness joints the unit compressive stresses may increase 25 to 35 percent over that at expansion joints because of the reduction in net cross section area due to the groove or bituminous parting strip employed at the pavement surface to form the joint.

Furthermore, observational studies on older pavements indicate that excessive localized stresses must occur at the adjoining faces of pavement joints or cracks because of the following acknowledged facts: (1) that the adjacent slabs are never in an ideal horizontal plane condition, due to the curling effect, and therefore the slab faces make contact at the upper or lower edges when the pavement is under compression, thus incurring high localized parasitic stresses which ultimately may result in spalling and abnormal compression failures; (2) that, in general formed joints are seldom constructed in a true vertical plane normal to the slab surface, and (3), the unequal distribution and character of infiltration material.

The high unit compressive stresses mentioned above are reduced at early ages of the pavement because of initial shrinkage of the concrete which is known to take place during the hardening process, and subsequently by stress relief due to elastic deformation and plastic flow of the concrete under continued compression. Studies by Clemmer (10)* indicate that the hardening shrinkage factor of concrete is approximately 0.02 percent. From the following relationship such a shrinkage factor would be equivalent to a temperature differential in the concrete of approximately 40°F:

$$T = \frac{d}{e} \quad \text{------------------------} \quad \text{XII}$$

where:

d = Linear displacement per unit length = .0002 inches

e = Coefficient of expansion = (.000005)

T = Temperature differential in degrees Farenheit

Thus the compressive stresses which would likely occur in a comparatively new pavement constructed without expansion joints and when subjected to the extreme temperature differential of 100°F would be in reality equivalent to those caused by a temperature differential of approximately 60°F. However, as the pavement increases in age there will be a progressive increase in compressive stresses. The slabs will become more and more restrained because of the gradual increase in slab lengths due to moisture changes and continued hydration, as well as by the infiltration of foreign matter at cracks and joints all of which tend to absorb the expansion space originally provided. Eventually the pavement becomes fully restrained.

However, when a concrete pavement becomes fully restrained with time, the maximum computed compressive stresses are not likely to materialize

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*(10) Clemmer, H. F., "Rigid Type Pavement Joints and Joint Spacing." Highway Research Board Proceedings, 1943.

because it is known that concrete under sustained loads undergoes a certain amount of plastic deformation which will reduce the compressive stresses. The amount of stress relief depends on how long the pressure is maintained.

Stress Relief by Plastic Deformation. According to Davis (11)*, for one test condition cited, the unit plastic flow in concrete reaches, under a unit stress of s = 600 p.s.i., applied at the age of 28 days, a value higher than 700 millionths per unit length after 2,000 days, 75 percent of which takes place within the first 100 days.

The elastic unit strain at s = 600 p.s.i., if $E = 5 \times 10^6$ p.s.i. is $\frac{600}{5 \times 10^6}$ = .000120 inches per unit length or approximately one sixth of that of maximum plastic flow.

According to the above, a concrete bar compressed to an initial stress of 600 pounds per square inch and constrained at the ends to a constant length will have at the end of 50 days a unit stress of approximately 100 pounds per square inch.

Now assume that a bar is fixed at the ends in a no-stress condition and when heated gradually will be gradually compressed. If there is no flow then the unit stress found by equation XI will reach a value of 2200 pounds per square inch for the following conditions:

$E = 5 \times 10^6$ p.s.i. (modulus of elasticity)

$e = 5.5 \times 10^{-6}$ (coefficient of expansion)

$T = 80°F$ (temperature differential)

If the temperature increase is gradual and extends over several months, plastic flow will occur and the resulting unit stress will be considerably smaller than 2200 pounds per square inch.

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*(11) Davis, R. E., Davis, H. E., and Brown, E. H.: "Plastic Flow and Volume Changes of Concrete". A.S.T.M. Proceedings, Vol. 37, Part II, 1937.

Therefore, applying the above results to highway pavements it may easily be seen that,

(1) The compressive stresses will probably never reach the values on which the pavement design is based.

(2) The warping stresses might be affected by flow to a smaller extent.

However, nothing conclusive can be said until such time as the effects of successive compressive and tensile plastic flows on the strength, crack expectancy and fatigue of concrete can be studied.

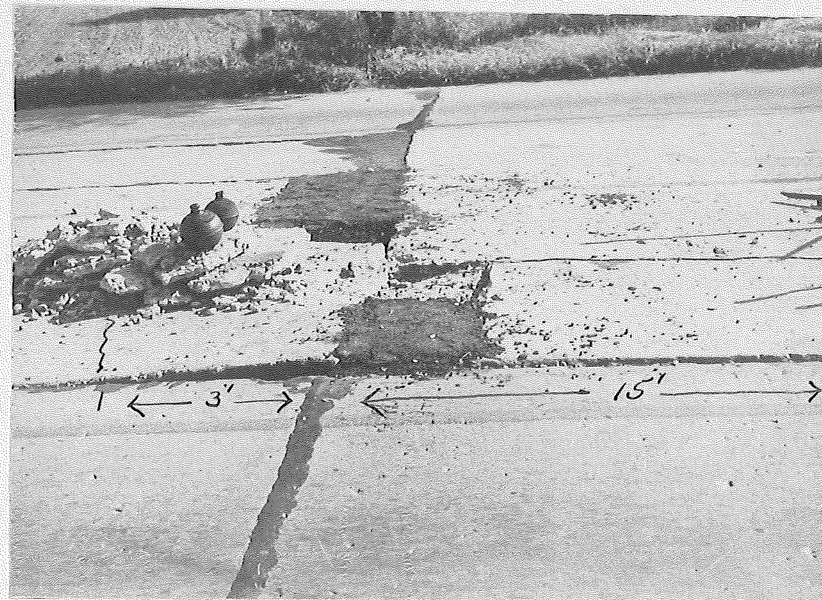
Compressive Failures. Failures in old concrete pavements due to lack of expansion space are generally manifested either by complete crushing of the weakened concrete adjacent to joint and cracks, or by the sudden eruption or blowup of fairly long sections of pavement. The two types of compressive failures are illustrated in Figure 11. The first type is fairly common in Michigan occurring on old pavements during prolonged periods of high temperatures. The second type is rare, no doubt because the common occurrence of the first type which offers sufficient stress relief to prevent blowups. Compression failures may be expected to be greatly reduced in the future because of improvements in expansion joint construction during the past 10 years such as the use of non-extruding joint filler and load transfer devices.

A recent survey of compressive failures occurring both in concrete pavements, and in old concrete pavements capped with a bituminous surface, which occurred in large numbers during the hot spell of 1944, revealed that the failure appeared, without exception, in pavements at least fifteen years old. Also it was observed that the compressive failures were of the first type occurring at cracks and joints where the concrete had become so weakened by stress fatigue and weathering that it could no longer withstand the compressive force. In such cases it was obvious that the compressive force could not be uniformly dispersed along both faces of the cracks or joints and that the effective net cross section of the slab had been materially reduced by spalling at both top and bottom edges. In the future such conditions are less likely to occur on modern pavements and employing the non-extruding type of joint filler provided they have been properly sealed and maintained throughout their life.

On the Michigan Test Road project, pavement sections up to 2700 feet in length were constructed with expansion joints only at the ends. The data presented in Figures 12 and 13 indicate that the movement of long sections due to temperature is confined to the end portions which in this case extend for a distance of approximately 900 feet from the ends. The 900 to 1000 foot lengths of pavement within the central portion of the same 2700 sections have been under restraint comparable to that of a pavement with no expansion joints for the last four years without any noticeable detrimental effect either on the central or end portions.

In addition, observational surveys throughout Michigan reveal that there are many miles of old concrete pavement which have been in full restraint for a number of years with no indication of compressive failures. Also it is generally recognized that a certain amount of compressive stress is considered desirable to prevent free cracking especially in long slabs.

In view of the above observations and since the normal compressive stresses cannot possibly exceed the value of 3500 p.s.i., as shown by calculations, even in the case of infinitely long distances between expansion joints, it may be concluded that, on straight level stretches, the expansion joints may be entirely omitted. Where the horizontal forces have to be considered, such as at structures, intersections, at sharp vertical or



A. Compressive Failure manifested by crushing of weakened concrete adjacent to joints and cracks. 44 G-25 (1)

B. Compressive Failure manifested by sudden eruption of a large section of pavement. 5-39F-7 (6B)

TWO TYPES OF COMPRESSION FAILURES. Figure 11

RELATIONSHIP BETWEEN SECTION MOVEMENT AND DISTANCE FROM CENTER OF SECTION

SERIES I-F – LENGTH 2700'
SPACING OF CONTRACTION JOINTS 60'

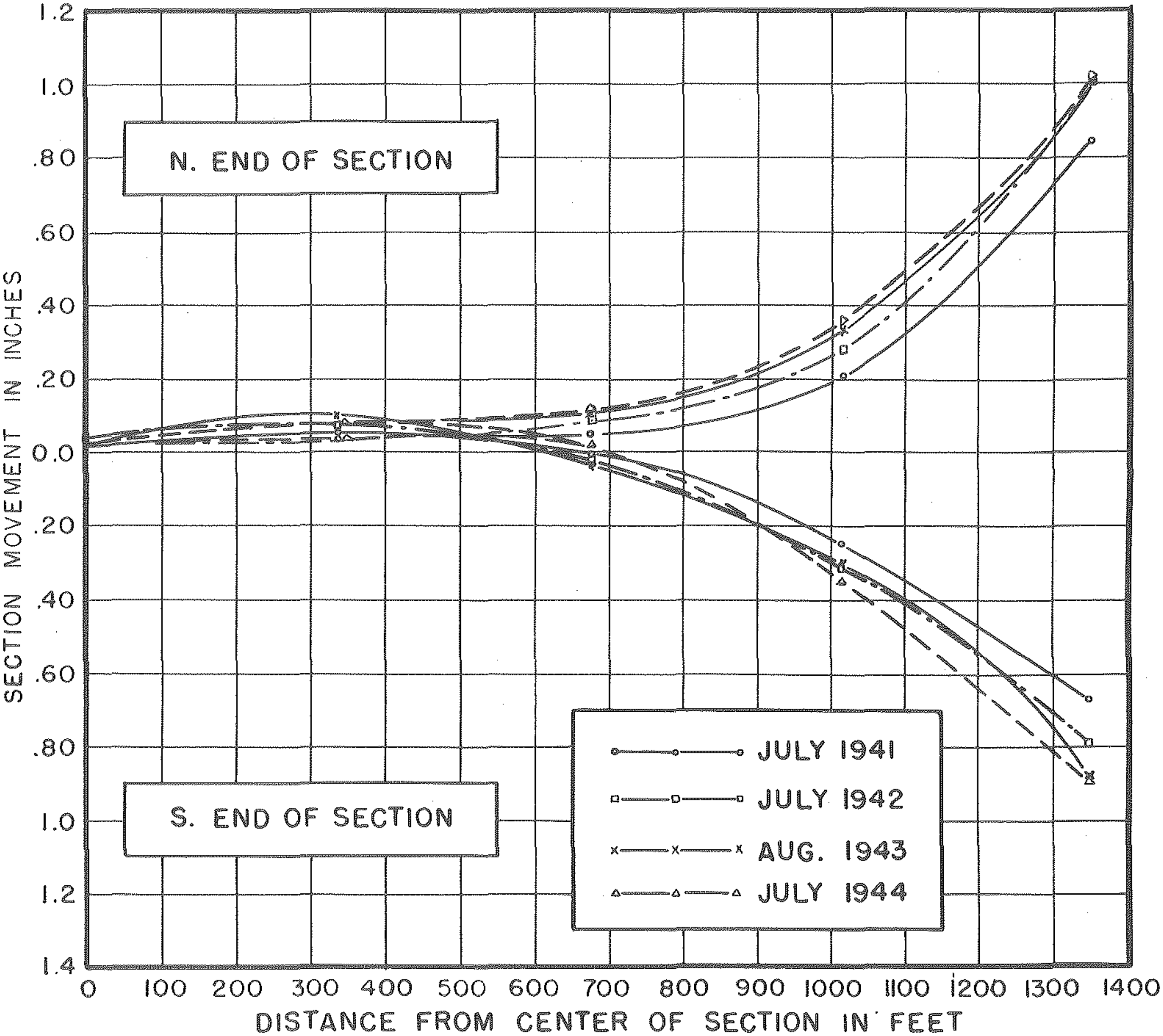


Figure 12

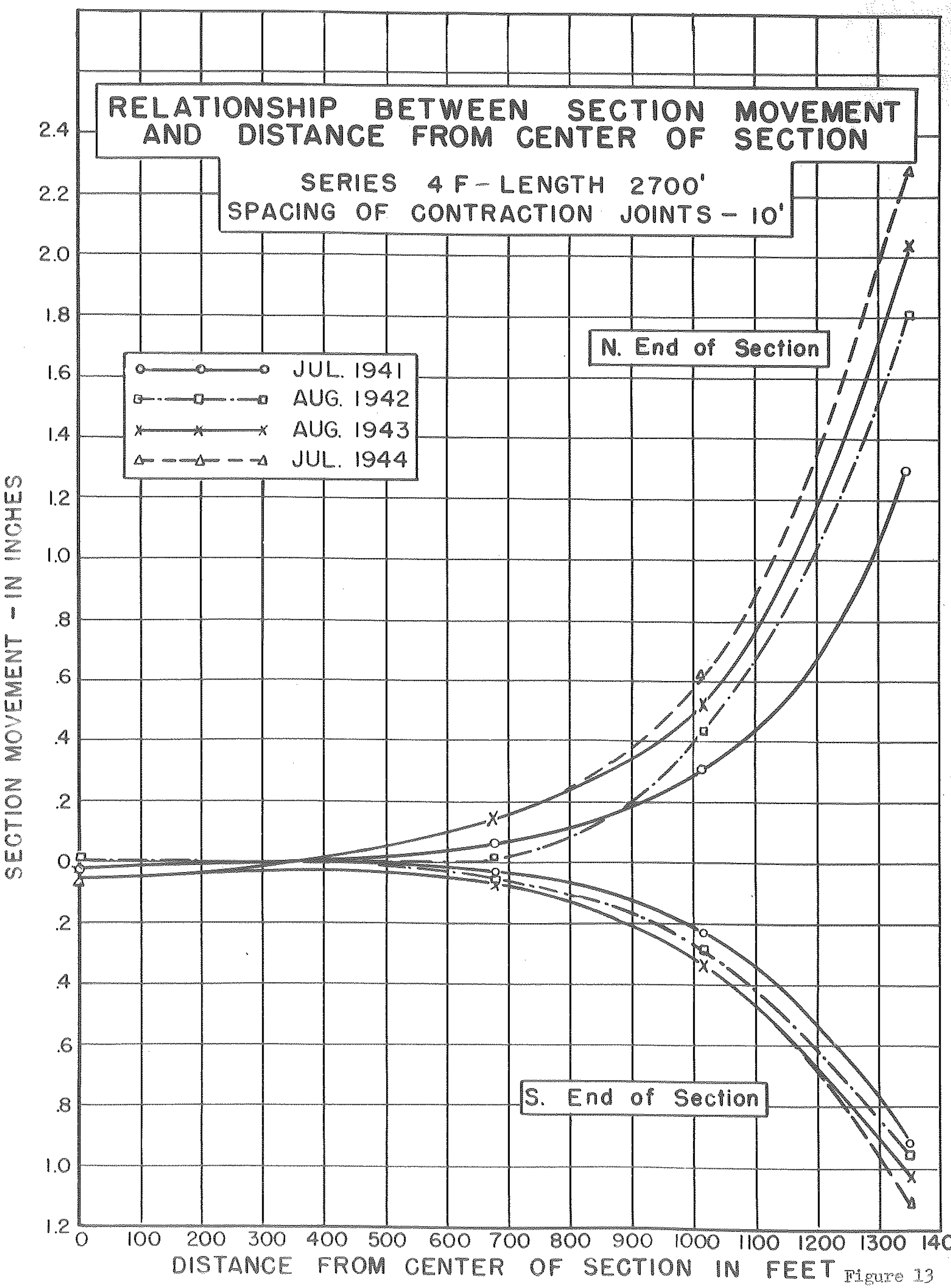

RELATIONSHIP BETWEEN SECTION MOVEMENT AND DISTANCE FROM CENTER OF SECTION
SERIES 4 F - LENGTH 2700'
SPACING OF CONTRACTION JOINTS - 10'
JUL. 1941
AUG. 1942
AUG. 1943
JUL. 1944
N. End of Section
S. End of Section
SECTION MOVEMENT - IN INCHES
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
.8
.6
.4
.2
0
.2
.4
.6
.8
1.0
1.2
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
DISTANCE FROM CENTER OF SECTION IN FEET

Figure 13

horizontal curves expansion joints should be provided.

Contraction Joints

Contraction plane of weakness joints are installed in the pavement for the purpose of controlling direct tensile stresses as well as warping stresses. Since the reinforcement must be broken at contraction joints, the joint is free to open, and, therefore, it is common ptractice to provide some kind of load transfer device at the joint. The common slip dowel is most generally employed for this purpose.

Contraction joint spacing will be governed by three factors; (1) The tensile and warping stresses in the concrete. (2) The allowable width of joint opening which must be consistent with good riding qualities, economic joint design and ability to exclude foreign matter and (3) the cost of steel reinforcement. Experience and research seem to indicate that approximately 100 feet is the maximum spacing for contraction joints which can be successfully employed at this time and meet the qualifications set forth above.

Warping Stresses Versus Slab Length. In order to present a general picture of the critical stress conditions that may result from restrained temperature warping, for different length slabs, stress diagrams have been prepared as shown in Figure 14, utilizing Westergaard's (12)* analysis of stresses in concrete pavements due to temperature. The curves in Figure 14 illustrate quite clearly, that for slab lengths less than 20 feet, the warping stresses diminish very rapidly, and for slabs 10 feet

(12)* Westergaard, H. M.: "Analysis of Stress in Concrete Pavements Due to Variations in Temperature." Highway Research Proceedings, 1926.

or less in length they become negligible. This fact has constituted the main reason for the trend towards shorter joint spacing, thus reducing free cracking because of lower combined stresses and thereby making possible reduction in slab thickness.

On the other hand, however, it is of further importance to note in Figure 14 that maximum warping stresses occur in slab lengths between 20 and 30 feet. Also for slab lengths in excess of 30 feet there is no material change in warping stresses at interior irrespective of slab length. Furthermore, in the interior portion of slabs longer than 30 feet, the stresses are somewhat less than those for slabs of shorter length, but increase slightly at a point approximately 13 feet from the slab ends. In the case of long continuous slabs approximately 20 feet or longer the warping stresses may be reduced and made comparable to those in the interior by making the contraction and expansion joints as rigid in bending as the concrete slab section is. This might eliminate the bulge in the stress curve near the ends of the slab. See Figure 14.

Joint Opening Versus Slab Length: It is an established fact that free contraction joints will open a slight amount shortly after construction due to normal shrinkage of the concrete during the initial hydration process, and subsequently to a greater degree during the first general cold spell after completion of the pavement.

The opening of plane of weakness joints due to contraction of the concrete during the hydration period may be influenced considerably by temperature and moisture conditions of the concrete at that time. Thus, by the preservation in the fresh concrete of the heat of hydration and

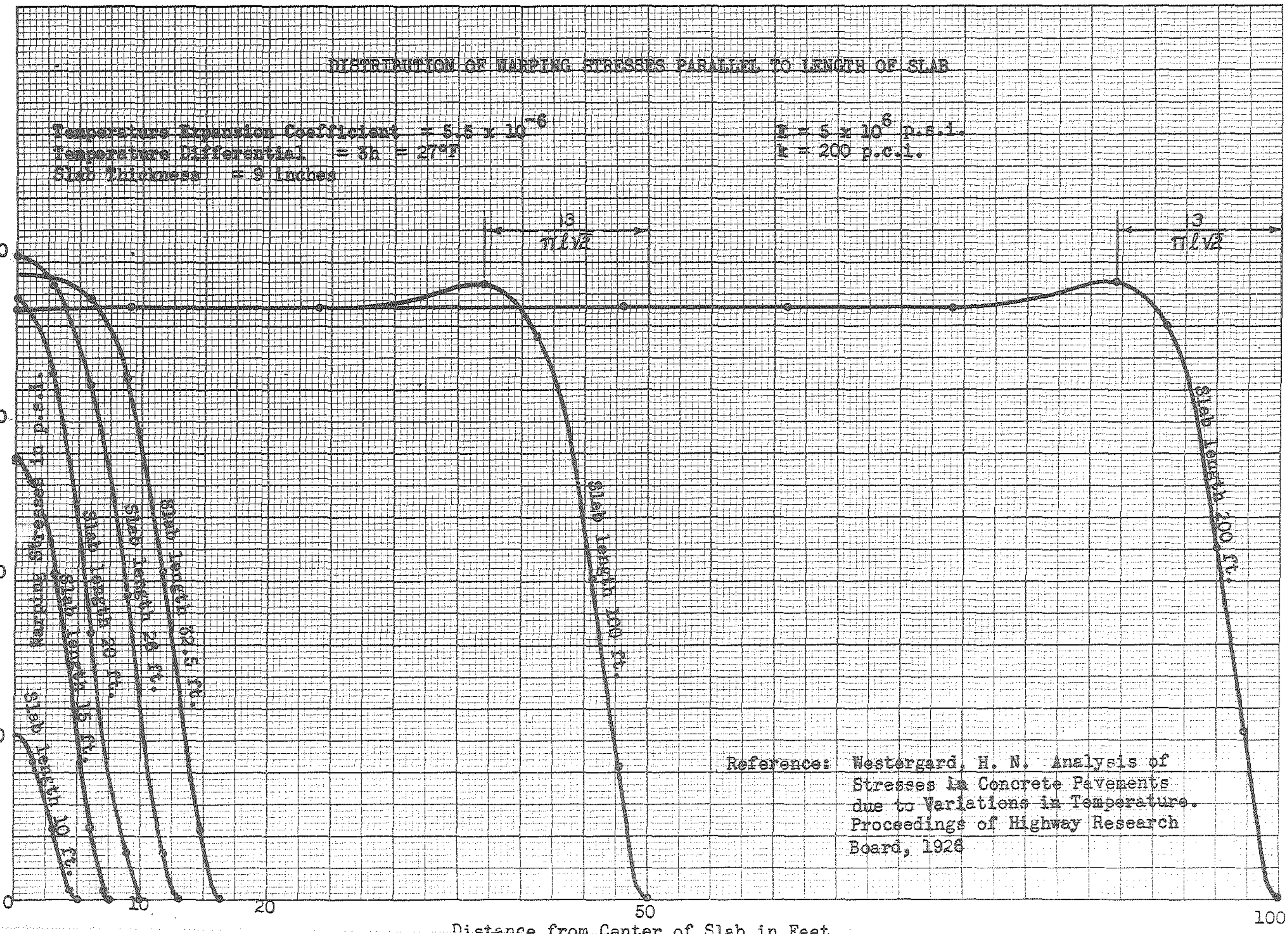

DISTRIBUTION OF WARPING STRESSES PARALLEL TO LENGTH OF SLAB
Temperature Expansion Coefficient = 5.5 x 10^-6
Temperature Differential = 3h = 27°F
Slab Thickness = 9 inches
E = 5 x 10^6 p.s.i.
k = 200 p.c.i.
13/πl√2
3/πl√2
Warping Stresses in p.s.i.
Slab length 10 ft.
Slab length 15 ft.
Slab length 20 ft.
Slab length 26 ft.
Slab length 32.5 ft.
Slab length 100 ft.
Slab length 200 ft.
Reference: Westergard, H. N. Analysis of Stresses in Concrete Pavements due to Variations in Temperature. Proceedings of Highway Research Board, 1926
0
100
200
300
400
10
20
50
100
Distance from Center of Slab in Feet

Figure 14

heat absorbed from other sources as well as moisture content (13)*, it might be possible to retard excessive tensile stresses during the critical period which exists at the early age of the pavement. Also, experiments with expanding concrete indicate the possibility of eliminating the formation of cracks at early ages of the pavement (14)*.

When transverse joints have once opened they seldom return to their original position even though the pavement is fully restrained. This is clearly illustrated by graphs in Figures 15-16-17-18 which show how the residual opening of contraction and dummy joints gradually increases with time. No doubt this condition is due mainly to the infiltration of foreign matter into the joint opening during periods of maximum opening. The increase in residual joint width opening with time is more pronounced as the spacing of contraction joints is increased and the slabs are not fully restrained due to the presence of expansion joints. This is also clearly illustrated by the above graphs resulting from slab movement studies included in the Michigan Test Road Project.

On the basis of the Test Road data it is logical to expect that the average contraction joint opening for slab lengths of 100 feet under full restraint will not exceed approximately 1/4 inch and, therefore, should not constitute a serious construction problem. However, in the case of 100 foot slabs partially unrestrained, such as will occur when they are located near expansion joints, it is to be expected that the joint opening will be greater than 1/4 inch.

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(13)* Swayze, M. A.: "Early Concrete Volume Changes and Their Control, A.C.I. Journal, Vol. 13, No. 5, April 1942.

(14)* Caquot, A and Lossier, Henry: "Expanding Cements and Their Application Self Stressed Concrete", LeGenie Civil VCXXI, No. 8, April 15, 1944. p. 61-65, and No. 9 May 1, 1944, p. 69-71. Abstract by R. L. Bertin. A.C.I. Journal, January 1945.

Cost of Reinforcement: A comparative study of concrete pavement costs, in which the proper weight of reinforcement is considered versus different contraction joint spacing and adequate joint construction for the same slab thickness, will show that the initial cost per mile of pavement with 100 foot contraction joint spacing is not materially greater than that when the contraction joints are spaced 20 feet apart. When contraction joints are considered at distances greater than 100 feet, the additional cost of reinforcement will probably overbalance the advantages gained by the use of fewer joints. However, the utilitarian advantages of constructing concrete pavements with as few joints as possible are obvious and this fact has not been overlooked in setting up the design recommendations.

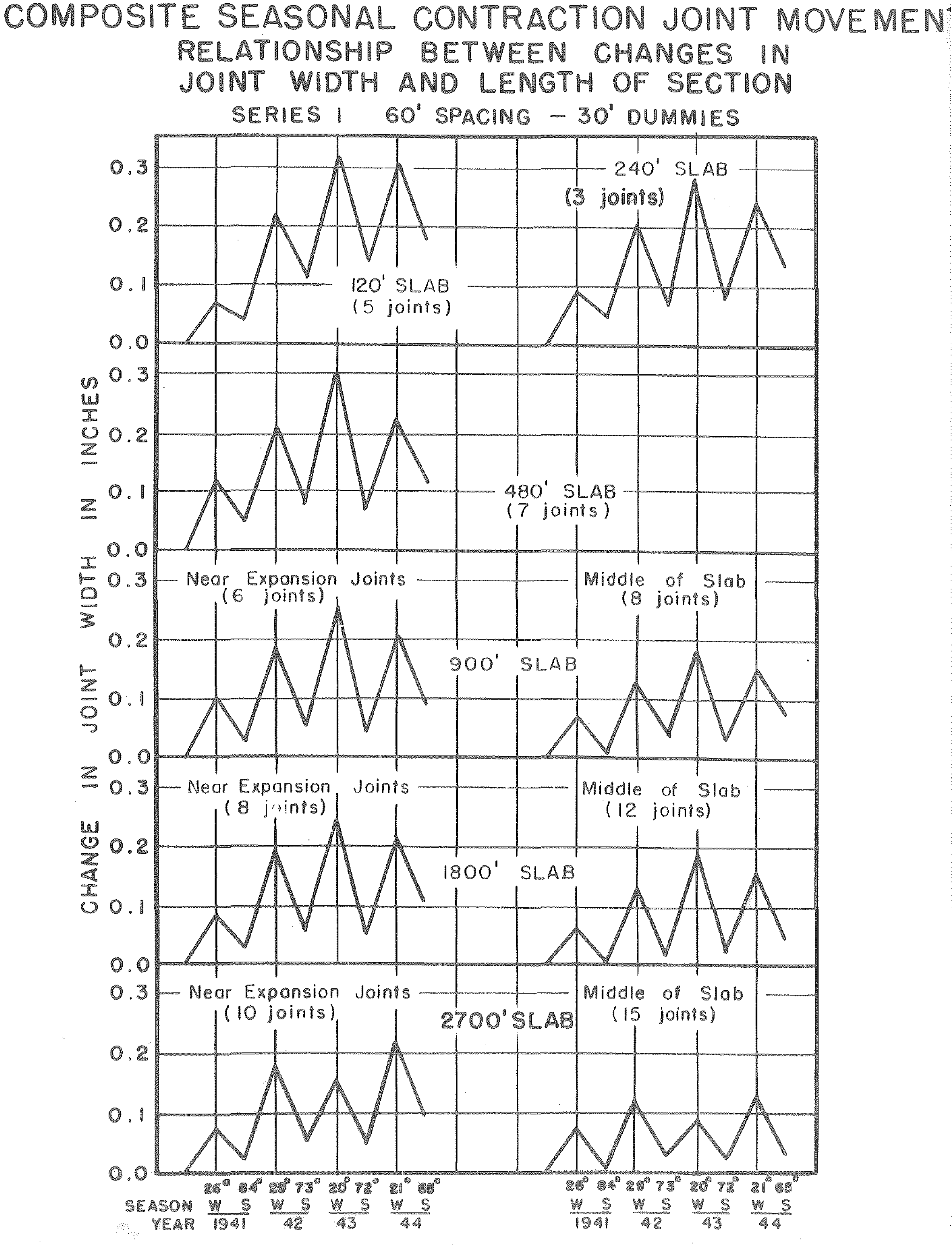
From 1924 to 1934 Michigan constructed many miles of concrete pavement with expansion joints only, which were spaced at 100 foot intervals. After more than 10 years in service many of these 100 foot slabs are in perfect condition, especially so in those areas where the pavements were constructed on well drained granular subgrade material.

Reinforcement

Steel reinforcement is considered necessary in concrete pavements to control free cracking and to prevent the opening of cracks when they occur.

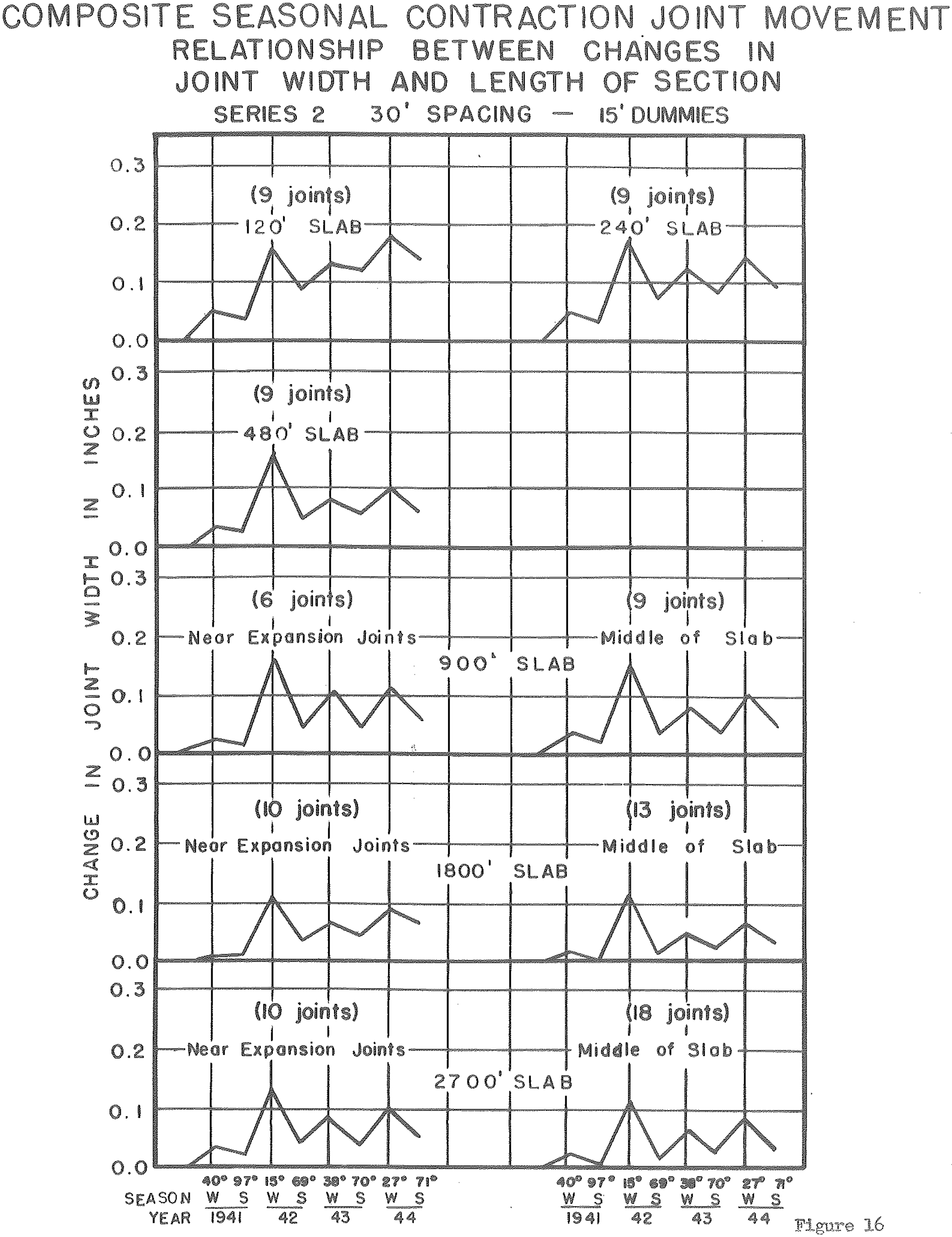
For a continuous slab the reinforcement should be dimensioned on the basis of methods used in reinforced concrete structures. However, since this method has proven impractical, reinforcement is now generally dimensioned for direct friction forces. This method seems to give satisfactory results and has been adopted in the present design.

Requirements for Longitudinal Steel Reinforcement. The amount of reinforcement necessary for proper design requirements can be readily obtained



COMPOSITE SEASONAL CONTRACTION JOINT MOVEMENT
RELATIONSHIP BETWEEN CHANGES IN
JOINT WIDTH AND LENGTH OF SECTION
SERIES I 60' SPACING - 30' DUMMIES
CHANGE IN JOINT WIDTH IN INCHES
0.0
0.1
0.2
0.3
120' SLAB
(5 joints)
240' SLAB
(3 joints)
480' SLAB
(7 joints)
900' SLAB
Near Expansion Joints
(6 joints)
Middle of Slab
(8 joints)
1800' SLAB
Near Expansion Joints
(8 joints)
Middle of Slab
(12 joints)
2700' SLAB
Near Expansion Joints
(10 joints)
Middle of Slab
(15 joints)
26° 84° 29° 73° 20° 72° 21° 65°
SEASON W S W S W S W S
YEAR 1941 42 43 44

Figure 15



COMPOSITE SEASONAL CONTRACTION JOINT MOVEMENT
RELATIONSHIP BETWEEN CHANGES IN
JOINT WIDTH AND LENGTH OF SECTION
SERIES 2 30' SPACING — 15' DUMMIES
CHANGE IN JOINT WIDTH IN INCHES
0.3
0.2
0.1
0.0
(9 joints)
120' SLAB
(9 joints)
240' SLAB
(9 joints)
480' SLAB
(6 joints)
Near Expansion Joints
(9 joints)
Middle of Slab
900' SLAB
(10 joints)
Near Expansion Joints
(13 joints)
Middle of Slab
1800' SLAB
(10 joints)
Near Expansion Joints
(18 joints)
Middle of Slab
2700' SLAB
40° 97° 15° 69° 38° 70° 27° 71°
SEASON W S W S W S W S
YEAR 1941 42 43 44

Figure 16

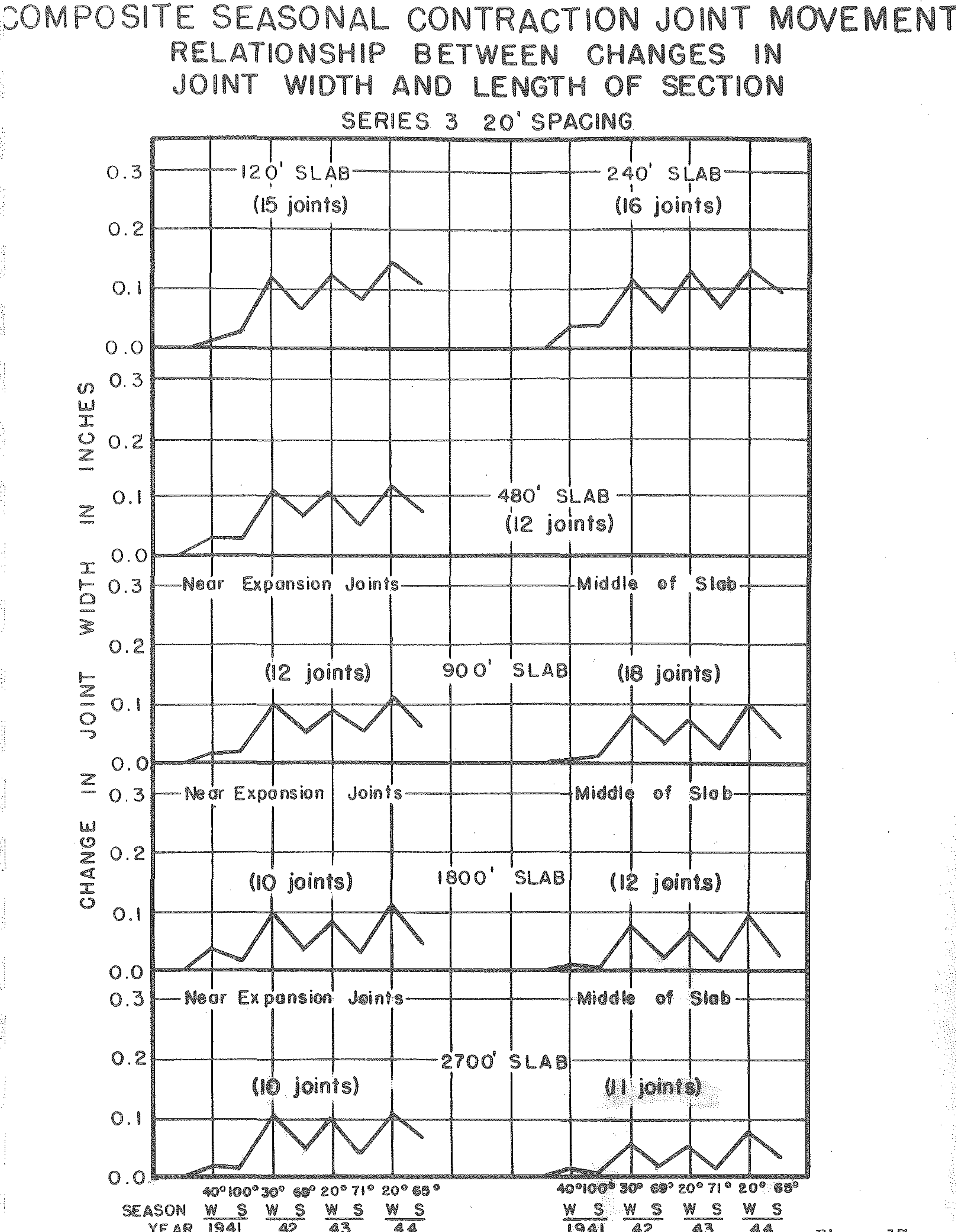

COMPOSITE SEASONAL CONTRACTION JOINT MOVEMENT
RELATIONSHIP BETWEEN CHANGES IN JOINT WIDTH AND LENGTH OF SECTION
SERIES 3 20' SPACING
CHANGE IN JOINT WIDTH IN INCHES
0.3
0.2
0.1
0.0
120' SLAB
(15 joints)
240' SLAB
(16 joints)
480' SLAB
(12 joints)
900' SLAB
Near Expansion Joints
(12 joints)
Middle of Slab
(18 joints)
1800' SLAB
Near Expansion Joints
(10 joints)
Middle of Slab
(12 joints)
2700' SLAB
Near Expansion Joints
(10 joints)
Middle of Slab
(11 joints)
40° 100° 30° 68° 20° 71° 20° 68°
SEASON W S W S W S W S
YEAR 1941 42 43 44
40° 100° 30° 68° 20° 71° 20° 65°
W S W S W S W S
1941 42 43 44

Figure 17

COMPOSITE SEASONAL CONTRACTION JOINT MOVEMENT
RELATIONSHIP BETWEEN CHANGES IN JOINT WIDTH AND LENGTH OF SECTION
SERIES 4 10' SPACING

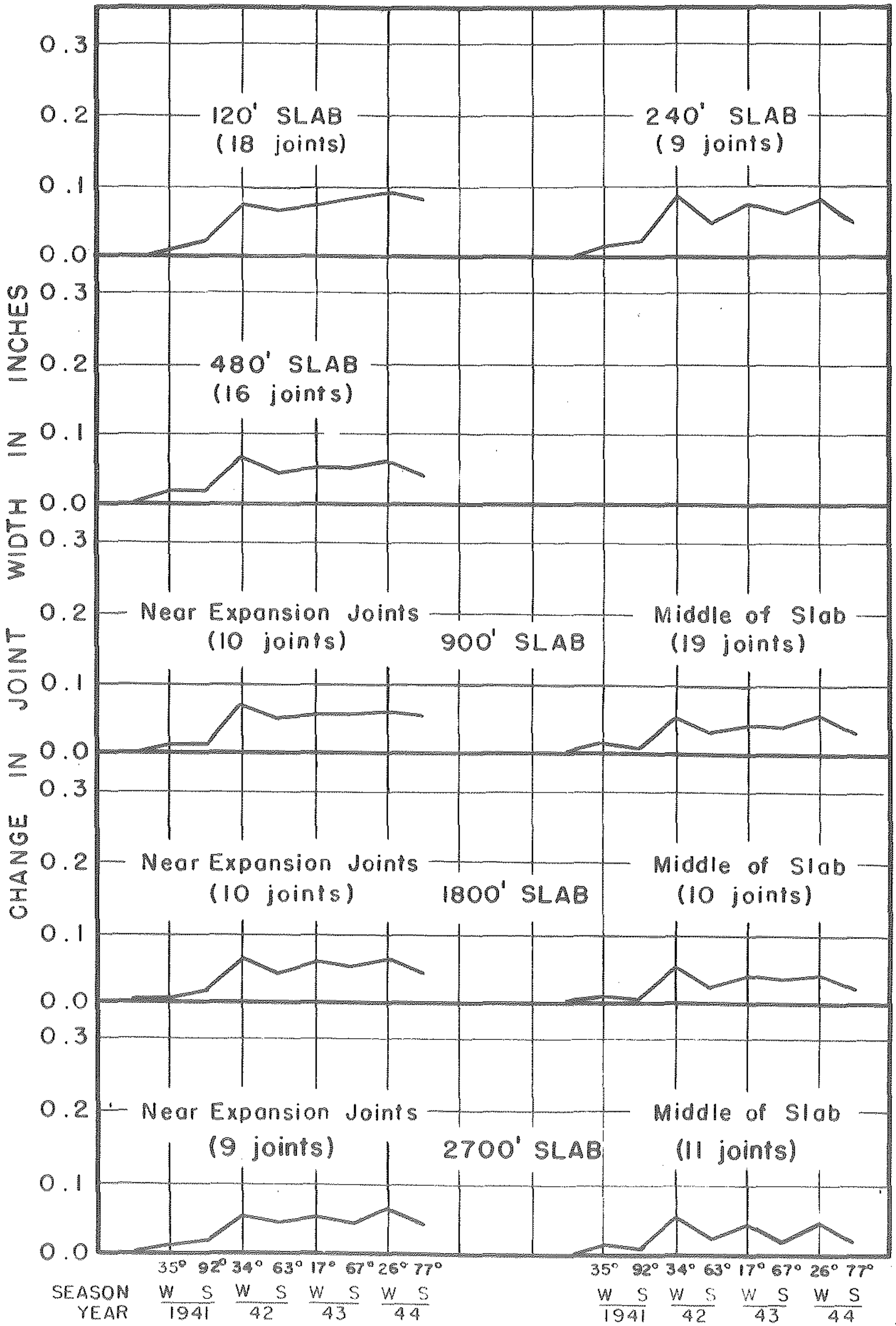
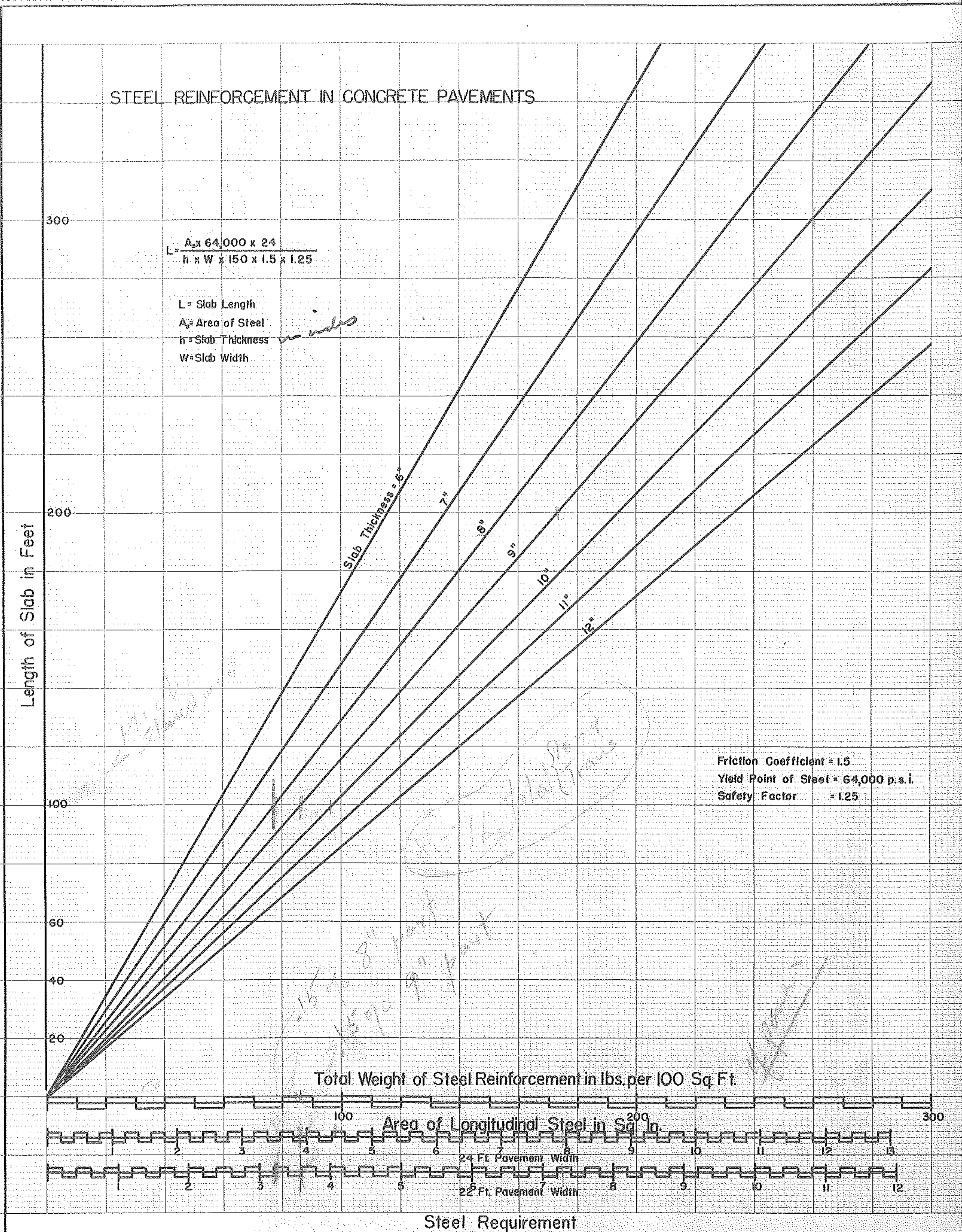


Figure 18



STEEL REINFORCEMENT IN CONCRETE PAVEMENTS
$L = \frac{A_s \times 64{,}000 \times 24}{h \times W \times 150 \times 1.5 \times 1.25}$
L = Slab Length
A_s = Area of Steel
h = Slab Thickness
W = Slab Width
Slab Thickness = 6"
7"
8"
9"
10"
11"
12"
Friction Coefficient = 1.5
Yield Point of Steel = 64,000 p.s.i.
Safety Factor = 1.25
Length of Slab in Feet
300
200
100
60
40
20
Total Weight of Steel Reinforcement in lbs. per 100 Sq. Ft.
100
200
300
Area of Longitudinal Steel in Sq. In.
1 2 3 4 5 6 7 8 9 10 11 12 13
24 Ft. Pavement Width
1 2 3 4 5 6 7 8 9 10 11 12
22 Ft. Pavement Width
Steel Requirement

Figure 19

from well-known relationships. As a matter of convenience in analyzing the joint spacing problem a graph has been prepared showing the relationship between slab length, slab thickness and quantity of reinforcement per 100 square feet of pavement.

The cross section area of longitudinal steel reinforcement for various slab lengths may be computed by the following equation:

$$L = \frac{As \times Ss \times 24}{h \times W \times d \times f \times C} \qquad \text{--------------------- XIII}$$

where:

L = Length of slab in feet

As = Cross sectional area of steel in square inches

h = Thickness of slab in inches

W = Width of pavement in feet

d = Unit weight of concrete, 150 pounds per cubic feet

f = Coefficient of concrete subgrade friction

C = Factor of safety = 1.25

Ss = Yield point of steel = 0.8 ultimate strength

Relationships between weight of reinforcement per 100 square feet and length and thickness of slab for a 22 foot pavement are presented in Figure 19. The values for reinforcement per 100 square feet in Figure 19 include present transverse steel requirements.

The graph which is given in Figure 19 is based on the Departments 1942 specification requirements for mesh reinforcement in accordance with A.S.T.M. Designation A 185.

PART III

JOINT DESIGN

The structural inadequacy of transverse joints has been fully appreciated by highway engineers for a great many years. At the present time the solution of the joint problem has not been fully attained and perhaps it never will be realized unless a conscientious effort is made to amend the following important factors which have exercised a profound influence upon the design, construction and performance of joints. They are:

1. The lack of a suitable joint filler and sealing compound.
2. The inability of present subbase construction to offer uniform and continuous support at pavement joints.
3. Until recently the comparatively slow progress in joint design because of the lack of informative data and sound design theories.
4. The lack of a recognized test procedure for evaluating the many proprietary load transfer devices on the market.

The Inadequacy of Present Joint Design

Observational studies of many miles of concrete pavements in Michigan, as well as in other states, reveal that considerable faulting has occurred at expansion and contraction joints constructed either with or without dowels or other types of load transfer devices. Pronounced pumping has been observed in Michigan on pavements located on trunklines carrying heavy commercial traffic. Also, vertical slab movement and faulting is becoming quite noticeable on wartime pavements in which the reinforcement and dowel bars have been eliminated as part of the war effort to save steel.

A cooperative study of the condition of transverse joints in concrete pavements was made during the summer of 1938 by the Michigan State Highway Department and the Public Roads Administration (15)* primarily to obtain information concerning the behavior of transverse joints

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(15)* Public Roads Administration: "A Cooperative Study of the Conditions of Transverse Joints in Concrete Pavements", Departmental Report by P.R.A., June, 1940 in concrete pavements in which the joints had been constructed without provision for load transfer. Conclusions from the survey indicate that faulting of the slabs at joints was prevalent in the older pavements examined in the survey, and to an undesirable extent on all pavements. It was also noted that faulting was evident in pavements constructed on all of the several types of subgrades, although pavements laid on clay-type subgrade materials were found to contain the greater percentage of badly faulted joints. Furthermore, from the evidence, it was concluded that some adequate structural connection between the ends of pavement slabs meeting at a transverse joint is an essential part of a good joint design, even though the slab ends themselves are structurally adequate.

A recent condition survey of series 10A and 10B of the design project (Michigan Test Road), which were established for the purpose of studying joint design, revealed considerable faulting at both expansion and contraction joints. The faulting was more pronounced in those sections constructed without dowels. These sections of pavement were constructed with and without 3/4" x 15" dowel bars. The survey data indicated that in the doweled Series 10A, 5 percent of the expansion joints had faulted 1/8 inch or more as compared to 67 percent in the undoweled Series 10B. Considering the contraction joints, in the doweled Series 10A, 8 percent had faulted 1/8 inch or more as compared to Series 10B, without dowels, where 32 percent of all contraction joints had faulted.

These above observed facts concerning the performance of transverse joints under various conditions of service indicate conclusively that our present design of concrete pavements is inadequate. It is realized that the majority of such failures may be primarily due to improper subgrade support at joints. Therefore, corrective measures could and should be considered in the preparation of the subgrade, either as a whole or in part, especially at joints. On the other hand, very much can be done to improve these conditions by the introduction of highly efficient load transfer units into joint construction and, in addition, by providing means of insuring proper sealing of the joints.

The design of transverse joints necessitates the consideration of certain structural features which enable the joint to perform its intended function, including, (1) allowance for movement of slabs during expansion or contraction, (2) provision for load transfer where necessary, (3) the insurance of mutual alignment of adjacent slabs to preserve desirable riding qualities, and (4) maintenance of adequate seal against infiltration of water and foreign matter. All of these features will be discussed in the following text.

Provision for Slab Movement

The amount of slab movement which will occur under normal conditions is dependent to a certain extent upon, (1) the length of slab or pavement section, (2) change in original slab length due to hydration and moisture, (3) seasonal and daily temperature fluctuations with respect to pouring temperatures, (4) subgrade characteristics, and (5) thermal coefficient of expansion of concrete. In all cases, it is desirable that the free joint opening be kept to a minimum. It has been shown in Part II that joint width movement can be controlled rather effectively by the proper spacing of expansion and contraction joints.

Expansion Joints: When expansion joints are considered solely for the purpose of controlling undesirable horizontal pressures, it is important to know how much expansion space should be provided to accomodate the expansion of different lengths of pavement. Studies of this character were incorporated into the Design Project of the Michigan Test Road. Throughout Series 1-2-3-4-11 and 12 monuments were placed at the ends of certain slab sections in order to observe the relative movement of sections of various lengths with respect to the subgrade. The total end movement in inches of the sections observed, as of July 1944, are given in Table XVI.

The data in Table XVI shows the relative movement of each end of the respective slab lengths as well as the average total movement obtained by averaging the end movements of sections of equal length. It is to be noted that the movement at the two ends of individual sections varies considerably in some cases. This is to be expected in the longer sections because, by virtue of their length, construction features may not necessarily be constant throughout the entire length of the section. Probably the general total residual movement of the adjacent sections in one direction or the other also influences to some extent the relative end movement of the intermediate sections. The unusually large movement indicated at station -3+65 at the northerly end of series 4F is considered to be due to the fact that there are 5, instead of 3, one inch expansion joints within a comparatively short distance to absorb the expansion.

The graphs in Figure 20 show the annual progressive changes in expansion joint widths associated with the slab movement study. The graphs indicate clearly how the sections tend to contract during the first winter after construction then undergo a considerable movement in the opposite

TABLE XVI

SUMMARY OF END MOVEMENTS OF SECTIONS OF VARIOUS LENGTH
OF
MICHIGAN TEST ROAD

Total End Movement in Inches as of July 1944

		S. E. End of Section			N. W. End of Section			
Series	Length	Monument	Temp. of Concrete F	Movement	Monument	Temp. of Concrete F	Movement	Total Average
				Design Project				
1A	120	771+20	72	0.304	772+40	76	0.284	0.588
1C	480	786+80	64	0.716	791+60	65	0.594	1.310
1D	900	807+20	65	0.713	816+20	66	0.976	1.689
1F	2700	236+60	68	0.888	863+60	75	1.119	2.007
2F	2700	864+20	69	1.141	891+19	69	1.172	2.313
3A	120	956+30	71	0.201	957+50	72	0.216	0.417
3D	900	990+30	70	0.650	999+30	70	0.926	1.576
3F	2700	1018+39	65	1.180	1045+36	75	0.917	2.097
4F	2700	1045+55	71	1.122	-3+65	78	*2.195	3.317
4D	900	19+40	78	1.224	28+40	79	0.657	1.881
4C	480	42+50	79	0.724	47+25	80	0.733	1.457
4A	120	60+67	80	0.369	61+87	80	0.492	0.801
				Durability Project				
11B	120	693+00	74	0.429	694+20	74	0.074	0.503
11C	360	694+30	74	1.247	697+92	75	0.488	1.735
11D	600	698+00	75	0.834	704+00	75	1.161	1.995
12B	120	729+00	69	0.342	730+20	69	0.035	0.377
12C	360	730+30	69	1.066	733+90	69	0.393	1.459
12E	600	736+52	69	0.478	742+52	69	1.392	1.870

Average of Total Section Movement

	Design Project				Durability Project		
Length of Slab in Ft.	120	480	900	2700	120	360	600
Expansion space provided	1"	2"	2"	3"	1"	2"	2"
Section Movement	0.588	1.310	1.689	2.007	0.503	1.735	1.995
	0.417	1.457	1.576	2.313	0.377	1.459	1.870
	0.816		1.881	2.097			
				3.317*			
Average	0.622	1.383	1.715	2.433	0.440	1.597	1.932

*Section end at south side of Muskegon River Bridge where there are 5 instead of 3 - - 1 inch expansion joints to absorb the expansion. This may account for large movement.

ANNUAL AND PROGRESSIVE CHANGES IN EXPANSION JOINT WIDTH

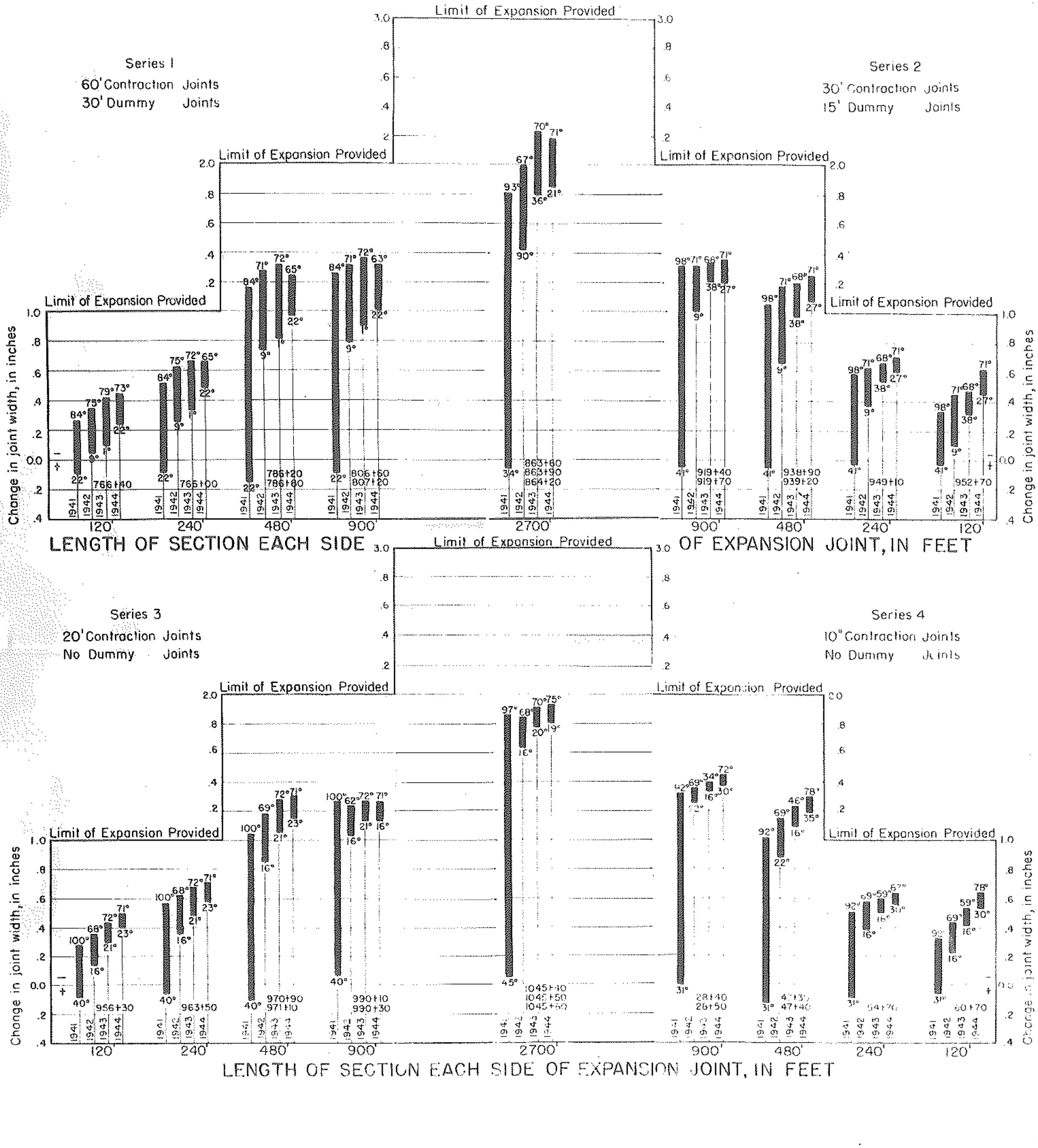


Figure 20

direction the following summer due to expansion. It is also noted that there is a slight additional lengthening of the sections with time. The effect of the spacing of intermediate contraction joints on the amplitude of movement at expansion joints is quite noticeable. The graphs further illustrate the relationship between spacing and width of expansion joints.

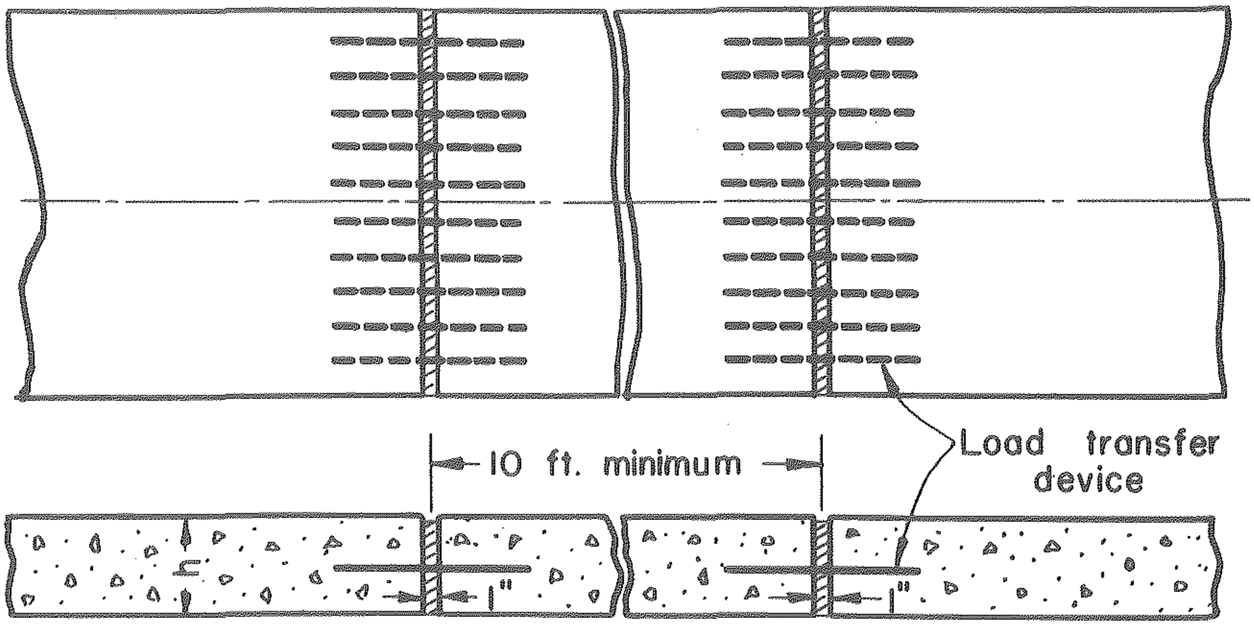
Furthermore, it has been demonstrated in Figures 12 and 13 of Part II that, in the case of the 2700 foot sections and for the subgrade conditions associated with the project, the movement of the section ends terminates at a point approximately 900 feet from the expansion joints. The intermediate portion of the pavement evidently remains stationary, and is therefore considered to be under full restraint. From this fact it may be concluded that, for similar circumstances under which normal expansion joints have been omitted throughout the project and similar horizontal stress relief is desired at a given location, it will be necessary to provide adequate expansion space for the movement of approximately 900 feet of pavement on each side of that location.

From the data presented above it would seem logical that a one inch expansion joint will provide sufficient expansion space for pavement sections up to approximately 400 feet in length which is being considered for fall construction starting September 15th. For conditions similar to those of the Test Road it is apparent that 2 to 3 inches of expansion space will be the minimum requirement at locations where horizontal stress relief is desired, such as would be the case when pavements are constructed without expansion joints.

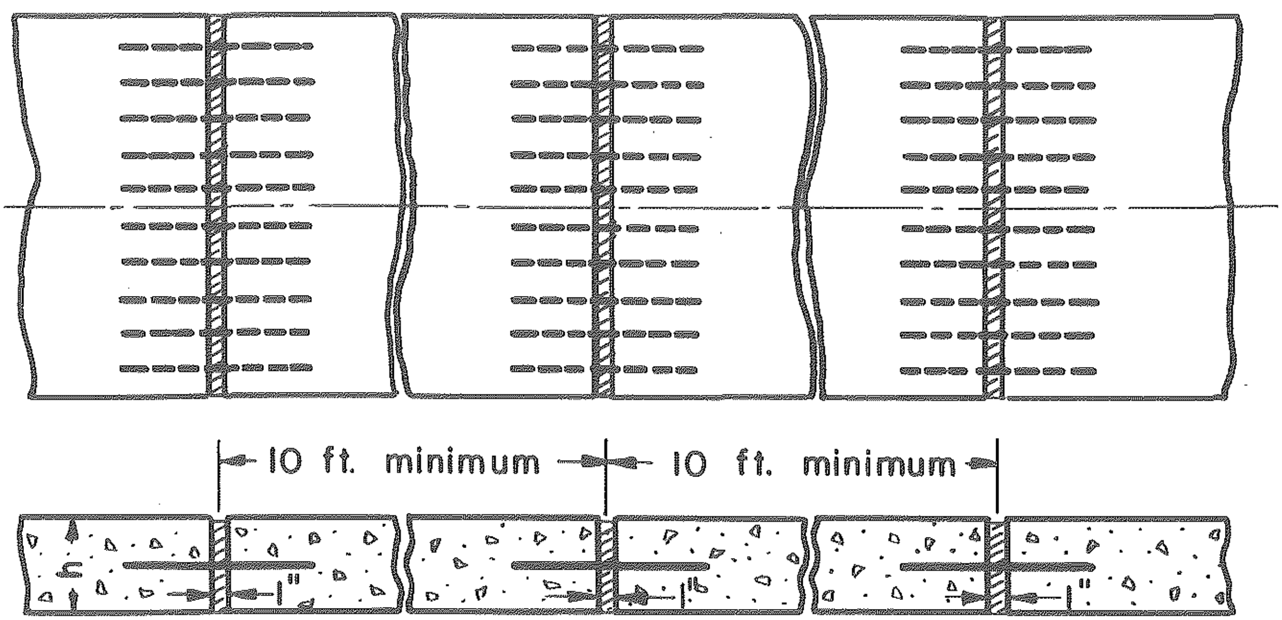
It is important to consider ways and means of controlling horizontal pressures. One method of handling this problem satisfactorily is by the installation of two or more on-einch expansion joints consecutively at short intervals. The shortest permissible distance for the interval between expansion joints should be 10 feet. Such a plan was employed on the Michigan Test Road and from all appearances the so-called "Expansion Relief Sections" are working satisfactorily. See Figure 21.

<u>Joint Filler</u>. Careful consideration should be given to the matter of providing the best type of joint filler material for expansion joints. Since the primary purpose of the joint filler is to prevent infiltration of foreign matter when the slabs are contracting and to support the joint sealing compound at the top, the material should have the greatest possible resilience combined with the property of minimum extrusion under cyclic pressures. The bituminous premolded joint fillers in common use today have excellent non-extruding properties, but on the other hand they possess low resilience, which is an undesirable feature. Wood boards have been used for years by some states with very good success. At the present time pre-compressed wood for joint filler material is beginning to receive favorable attention. It is understood that the wood boards are pre-compressed in the dry state to approximately 70 percent of their original thickness, and are then inserted in the pavement while still in this condition. Because of its pre-compressed condition the wood joint filler has a potential swelling distance of approximately 30% of its original width plus the distance it would normally swell under saturated conditions. It is assumed that the pre-compressed wood when fully saturated will expand sufficiently to keep the expansion joint opening completely closed, or nearly so, at all times.

<u>Contraction Joints</u>. In order to insure cracking of the pavement at contraction joints so that each joint may function as intended, it is



EXPANSION RELIEF - TYPE 2



EXPANSION RELIEF - TYPE 3

EXPANSION RELIEF FOR LONG PAVEMENT SECTIONS

necessary to weaken the pavement by reducing its net cross-section area at the joints. This is accomplished by either grooving the surface of the pavement or inserting a narrow premolded bituminous parting strip in the top of the pavement. Experience indicates that a reduction of 2-1/2 inches in thickness is sufficient to insure cracking at contraction joints for pavements up to 10 inches in thickness.

On the basis of many observations, it is obvious that the bituminous premolded parting strip so commonly used to create the contraction joint not only provides inadequate seal to the contraction joint when open, but because of its flexibility and instability it is difficult to install properly. Consequently spalling and subsequent disintegration are quite common when this material is used. See Figure 22. The grooved type of joint when properly sealed with an asphaltic-rubber compound offers many more advantages.

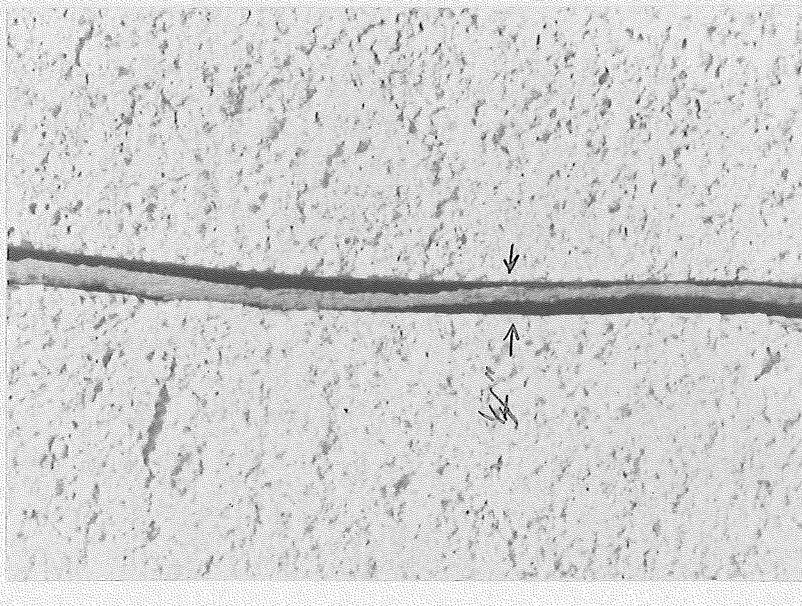
Observational studies of cracking at contraction joints reveal that some device is desirable at the bottom of the pavement as well as at the top to insure cracking in a vertical plane throughout the joint. It has been observed that a narrow parting strip installed at the bottom of the pavement in the same vertical plane as the groove at the surface is a satisfactory method to control cracking at contraction joints.

As illustrated in Figure 22, contraction joints under present circumstances present a definite maintenance problem, especially when they open more than 1/4 inch. The longer the slabs are constructed, the wider will be the joint opening and the more serious the problem becomes.

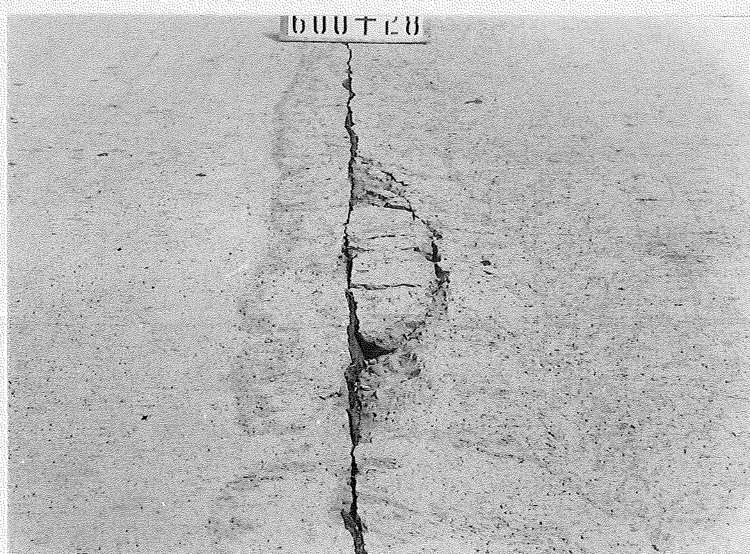
To throw some light on the matter of joint width movements which might be expected for 100 foot continuous slab lengths as proposed in the design analysis, the joint width movements of several 100 foot continuous slabs constructed in Series 9 of the Michigan Test Road have been presented in Table XVII . One inch expansion space was provided between consecutive slabs. The slabs are of 9"-7"-9" construction with no steel reinforcement or intermediate joints, and rest upon a sand subbase with a friction coefficient of approximately 1.0 to 1.5. The average air temperature at time of pouring was 70°F. The slabs were maintained under a horizontal pressure of 200 pounds per square inch for a period of 3 days. The pressure was applied gradually, starting 4 hours after pouring, until it reached a maximum at approximately 16 hours after pouring. From these data it is apparent that at least a 1/2 inch joint width movement can be expected of slabs 100 feet long and, therefore, provisions will have to be made to prevent the infiltration of undesirable materials.

As a matter of further interest in this study, the joint width movement of several expansion joints in Series 9 have been plotted in Figure 23 to show their behavior since construction. These graphs in Figure 23 bring out several interesting facts: (1) the slabs show remarkable similarity in their seasonal movements, (2) the bulk of the slab movement is in the nature of contraction, and, since the slabs were poured at a fairly high temperature, this fact brings out the influence of pouring temperature on future slab behavior. These joints are essentially contraction joints since very little movement takes place beyond that of the original pouring position, which is considered the point of zero movement.

It would appear then that the time of year at which a pavement is poured would determine whether a joint will function primarily as an

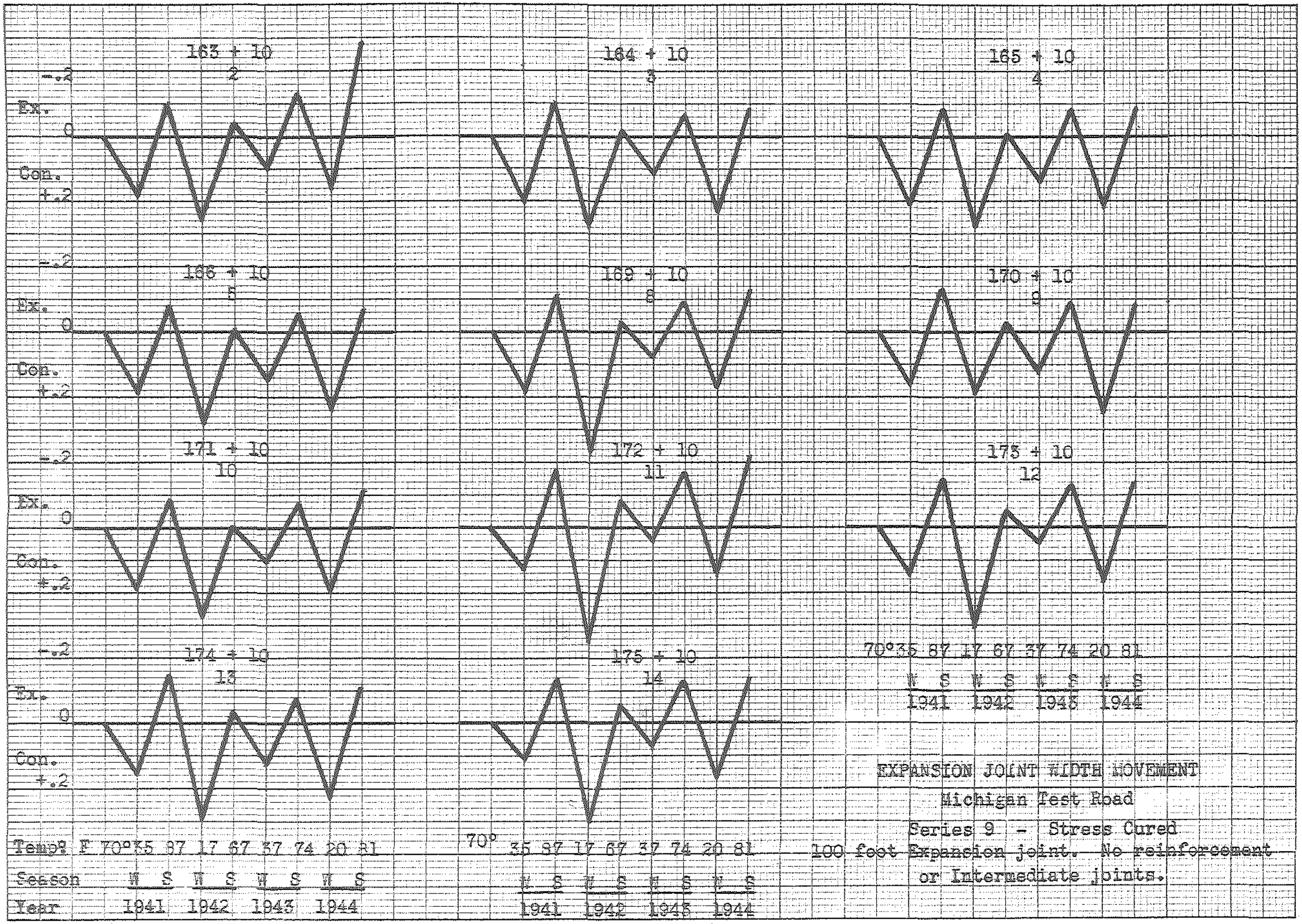


A. Typical Condition of Contraction Joints During Winter Time.



B. Typical Spalling at Plane of Weakness Joints.

TYPICAL CONTRACTION JOINT WEAKNESSES. Figure 22



EXPANSION JOINT WIDTH MOVEMENT
Michigan Test Road
Series 9 - Stress Cured
100 foot Expansion joint. No reinforcement or Intermediate joints.

Figure 23

TABLE XVII.

SUMMARY OF EXPANSION JOINT WIDTH MOVEMENT

*SERIES 9, MICHIGAN TEST ROAD

Joint No.	Station	Temp. 35°F	Temp. 17°F	Temp. 37°F	Temp. 20°F	Temp. 87°F	Temp. 65°F	Temp. 74°F	Temp. 81°F
2	163+10	+.185	+.263	+.101	+.167	-.106	-.045	-.135	-.284
3	164+10	.212	.285	.124	.238	.100	.016	.065	.090
4	165+10	.216	.280	.144	.220	.080	.002	.080	.094
5	166+10	.189	.284	.125	.244	.082	.005	.060	.076
8	169+10	.187	.379	.081	.168	.108	.032	.096	.132
9	170+10	.167	.196	.126	.248	.133	.035	.087	.087
10	171+10	.192	.478	.114	.198	.096	.000	.068	.108
11	172+10	.134	.347	.040	.145	.167	.091	.169	.219
12	173+10	.147	.310	.053	.169	.144	.055	.133	.141
13	174+10	.160	.292	.133	.231	.146	.040	.074	.105
14	175+10	.118	.302	.071	.166	.140	.058	.124	.137
Average Movement		+.173	+.310	+.101	+.199	-.118	-.034	-.099	-.134

Winter	+	.173	.310	.310	.101	.101	.199	.199
Summer	-	.118	.118	.034	.034	.099	.099	.134
Total Movement		.291	.428	.344	.135	.200	.298	.333
To nearest 1/16"		5/16	7/16	6/16	2/16	3/16	5/16	5/16

*Series 9 - Stress Cured, 100 foot expansion joints, no intermediate joints, no reinforcement.
Load transfer at expansion joint.
Average air temperature at pouring 70°F.
Sand Subbase material, coefficient of friction approximate 1.0 to 1.5.

expansion or a contraction joint. This being true it should be possible to govern the type of joint construction accordingly.

For such widths of contraction joint openings as may be expected for 100 foot slabs, it will also be necessary to provide special means for preventing infiltration of foreign matter and water at the sides and bottom of the joints. The method suggested for consideration is the employment of a properly designed metal shield of sufficient width to completely seal the sides and bottom of the joint to prevent infiltration, the metal shield to be constructed as an integral part of the dowel bar holding assembly. The narrow parting strip, mentioned previously as being desirable at the bottom of the joint to insure vertical cracking, may be attached to the metal shield to facilitate installation.

Tests by the Department on the effectiveness of aggregate interlock disclosed the fact that the efficiency of aggregate interlock as a load transfer medium decreases very rapidly with increase of joint width opening, and that it cannot be depended upon for joint width openings greater than approximately 1/16 inch. Since measurements on the test road show contraction joint openings, even on restrained pavements, of as much as 1/4 inch in winter, it is obvious that aggregate interlock should be given no consideration as a load transfer mechanism in joint design. The Public Roads Administration (16)* in a recent report states that "0.04 inch is the maximum width opening that aggregate interlock can be depended upon."

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(16)* Sutherland, E. C. and Cashell, H. D.: "Structural Efficiency of Transverse Weakened-Plane Joints." Highway Research Board Proceedings, 1944.

In addition to providing for slab movement, a joint and its environs should preserve indefinitely the original riding qualities of the pavement. Experience indicates that in order to attain this end, it is necessary, as an insurance measure, to provide a suitable mechanism capable of the two-fold function of transferrring the load across the joint and maintaining the mutual alignment of the adjoining slab surfaces. By the introduction of a suitable device the total mutual deflection of the slab ends at joints may be reduced 50 percent and the deflection of one slab relative to the other almost 100 percent. Thus, in the presence of reduced subgrade support in the vicinity of joints, which is certain to occur unless steps are taken to correct the subgrade, the mechanism will aid greatly in the prevention of pumping and subsequent faulting at joints. The mechanisms so employed are commonly referred to as load transfer devices or joint units.

Load Transfer Methods and Slab Alignment

If pavements, including surface and subgrade, are designed and constructed in accordance with the principles and assumptions set forth at the beginning of this report without consideration of load transfer, then no provision for load transfer across the joints should be necessary. However, the design analysis for slab thickness does not take into consideration non-uniform subgrade conditions, subgrade volume changes caused by moisture or frost action, nor the pumping action and subsequent faulting of the slab ends resulting from decreasing subgrade support in the vicinity of joints. This lack of stability in subgrade support at joints is due to three discernible factors: (1) continued consolidation of granular subgrade material by vibration caused by the hammering effect of traffic over the joint edges, (2) by the diluting effect of surface water which filters into these areas through open joints, or by seepage at the sides of the pavement and (3) plastic deformation of the subgrade under dynamic load action due to the fact that the deflection of the slab ends may be 3 to 5 times that of the interior of the slab.

Load transfer as a factor in joint design was first recognized in 1917 when, on a concrete pavement project near Newport News, Virginia, steel dowels were placed across all transverse joints for the stated purpose of transmitting load across the joints by shear. Since that time numerous patents have appeared covering all kinds of imaginable schemes for constructing transverse joints. In most cases they were so expensive or impractical to construct that they never have been considered (17).

Load Transfer Practices. Although methods to provide load transfer or structural interaction of adjoining slabs were used as early as 1917, no attention was given to the determination of spacing or to the evaluation of the effectiveness of such devices until 1928. In that year H. M. Westergaard (18) presented a rational theory for joint design under the title, "Spacing of dowels". Since that time several investigators (17, (19), (20), (21) have presented theories and methods of test for the evaluation of load

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(17) Teller, L. W. and Sutherland, E. C.: "A Study of the Structural Action of Several Types of Transverse and Longitudinal Joint Designs." Public Roads, Vol. 17, No. 7, Sept. 1936 and Vol. 17, No. 8, Oct. 1936.

(18) Westergaard, H. M.: "Spacing of Dowels" Proceedings of Highway Research Board, 1928.

(19) Kushing, J. W. and Fremont, W. O.: "Joint Testing Experiments with a Theory of Load Transfer Distribution Along the Length of Joints." Proceedings of Highway Research Board, Dec. 1935.

(20) Kushing, J. W. and Fremont, W. O.: "Design of Load Transfer Joints in Concrete Pavements." Proceedings of Highway Research Board, Nov. 1940.

(21) Friberg, B. F.: "Dowel Design." Proceedings of A.S.C.E. Nov. 1939.

transfer devices together with results of field and laboratory studies.

The results of these investigations, particularly in regard to the functioning of load transfer devices, indicate that there exists, in some cases, a close agreement between field tests and values obtained by theoretical calculations based on the Westergaard theory of slab deflections. A thorough study of these investigations is encouraging and gives evidence that a rational method of joint design can be established whereby a definite relationship between load transfer device characteristics and the characteristics of slab and subgrade may be utilized.

An examination of the load transfer devices used at the present time reveals certain definite principles in their design. These principles of design may be classified as follows:

1. The development of shear resistance with hinge action.
2. The development of shear and bending moment.

A classification of several well-known load transfer devices with respect to the above principles is given in Table XVIII.

As a matter of interest, Table XIX has been prepared in which the relative rigidities of different load transfer units spaced on 15 inch centers are compared with that of a 10" x 15" concrete beam simulating a section of pavement. The data in Table XIX indicate that in order to make a joint unit as rigid as the slab, it would be necessary to use the equivalent of steel WF Sections 8" x 8" - 58# at 15 inch centers which is impractical. The data also show that such a slab action has a rigidity approximately 13,600 times that of a common 3/4 inch dowel bar on 15 inch centers.

<u>Load Transfer Practice in Other States</u>. The development of load transfer practice in Michigan is no doubt typical of that which has

TABLE XVIII

CLASSIFICATION OF SOME WELL KNOWN LOAD TRANSFER DEVICES

Classification	Trade Name	Manufacturer
1. Shear with hinge action	Translode Base	Highway Steel Products Co. Chicago Heights, Illinois
	Translode Angle Unit	" " "
	Keylode Const. Joint	" " "
	The "Acme" or Ahles	Acme Steel & Malleable Iron Works, Buffalo, N.Y.
	Flexible Dowel	" " "
2. Shear and Bending Moment	Conventional Dowel Bars	
	Double Dowel Unit	Highway Steel Products Co. Chicago Heights, Illinois
	Godwin Joint	W. S. Godwin Co., Inc. Baltimore, Md.
	Tee-Gee Bar	Five Way Expansion Joint Co., Chicago, Illinois
	J-Bar Unit	American Concrete Expansion Joint Co. Chicago,Ill
	Nat'l Road Joint Unit	Nat'l Road Joint Co., Chicago, Illinois

TABLE XIX

COMPARISON OF JOINT UNIT RIGIDITY VERSUS CONCRETE SLAB

Designation of Joint Units	I $in.^4$	E.I. lb. $in.^2$	Relative Rigidity
Concrete Beam 15"wide, 10"deep	1.25×10^3	6.25×10^9	13.60×10^3
Am. St. Beams. 12" x 5" - 31.8#	215.8	6.47×10^9	14.08×10^3
WF Section 8" x 8" - 58#	227.3	6.82×10^9	14.82×10^3
Steel Bar 0.5" wide,2"deep	0.333	10^7	21.75
Steel Bars			
D = 1.62"	0.333	10^7	21.75
D = 1.25"	0.119	3.57×10^6	7.77
D = 1.00"	0.049	1.47×10^6	3.2
D = 0.75"	0.0154	0.46×10^6	1.00

Modulus of Elasticity for Concrete $E_c = 5 \times 10^6$ p.s.i.

Modulus of Elasticity for Steel $E_s = 30 \times 10^6$ p.s.i.

taken place in other states. For example, a study of the data in Table XX , which was compiled from the 1941 Summary of Concrete Roads Specifications prepared by the Portland Cement Association, reveals the great diversity of opinion as to what constitutes adequate load transfer for transverse joints in concrete pavements.

A further analysis of the data in Table XXI brings out several interesting facts concerning load transfer practice at the beginning of the war:

1. Conventional 3/4 inch dowel bars are used to a greater extent than any other type of load transfer device.
2. In some instances 7/8 inch and 1 inch dowel bars are being used instead of the 3/4 inch size.
3. Dowel bar lengths vary from 6 inches to 27 inches. The majority being either 15 inches or 24 inches in length.
4. The spacing of dowel bars is consistent with the requirements of the Public Roads Administration, that is, 12 inch to 15 inch centers. Other devices are the same.
5. Several states permit the use of certain approval proprietary types of load transfer devices for expansion joints and dowel bars for contraction joints.
6. Some states prefer to use load transfer devices of their own special design.

This data has been presented to show the disagreement which exists in design requirements for load transfer across transverse joints and to further prove the need of a method for evaluating load transfer devices on the basis of their ability to perform their intended function.

TABLE XX

LOAD TRANSFER METHODS AS EMPLOYED BY STATE HIGHWAY DEPARTMENT*

Dowel Size & Spacing	Other Types At Expansion Joints	Other Types At Contraction Joints	States Using These Types
3/4"x24"x12"	N	N	Colo., Del., Idaho, Mont., Nev., N.M., Vt., W. Va.
3/4"x24"x12"	N	D	N.D.
3/4"x24"x12"	A	A	R.I., Va.
3/4"x24"x12"	D	AI	Oregon (AI=Aggregate Interlock)
3/4"xNSx12"	N	N	Me., Penn., Utah
3/4"x15"x12"	N	N	D. of C.
3/4"xNSx12"	TN	D	Fla. (TN=Translode Nat'l Dowels)
3/4"xNSx12"	App.	App.	Ind. (App.=Any approved unit
3/4"xNSx12"	Alt.	Alt.	Ky. (Alt=Approved load-transfer unit)
3/4"xNSx12"	D	D	Md.
3/4"x12"x13"	T	N	S.C.
3/4"x24"x13-1/2"	Wor T-OC	N	Ill. (W=Wing anchor)
3/4"x21"x14"	N	N	Tex.
3/4"x27"x15"	A	D#	Neb. (#Dowel length 24")
3/4"x24"x15"	N	N	La.
3/4"x24"x15"	SP	SP	Wis.
3/4"x24"x15"	T	N	Ga.
3/4"x15"x15"	N	N	Ala., Ariz., N.C., Tenn.
3/4"x15"x15"	SP	D or G	Mich.
3/4"x15"x15"	T or DW	T or DW	Ohio (DW=Dow-weld)
3/4"x15"x15"	N	D	Minn. (7/8"ø for 9-7-9)
3/4"x14"x15"	N	N	Calif.
3/4"x16"x15"	N	N	Miss.
3/4"x22"x14"	N	D#	Ia. (8-7-8) (#24" long at 12"cc)
3/4"x24"x12"	N	D	Ia. (other than 8-7-8)
7/8"x24"x12"	T-OC	N	Kan.
7/8"x16"x12"	TA-OC	D or TA	Mo. (TA=Translode angles)
7/8"x13-1/2"x12"	N	D#	Wash. (#3/4x12x15 on FAP)
1"øx16"x15"	A	N	Ark.
1-1/2"xøx12"	D	N	Conn.
10 ga. triang. at 15"	B	N	N.H. (B=Bethlehem type or equal)
2"x1/2"x7/8" ch.at 12"	N	N	N.J.
Various	Various	N	N.Y.
A	A	N	Mass.
A	A	D#	Okla. (#3x4"øx24"x12")
A	N	N	S.D., Wyo.

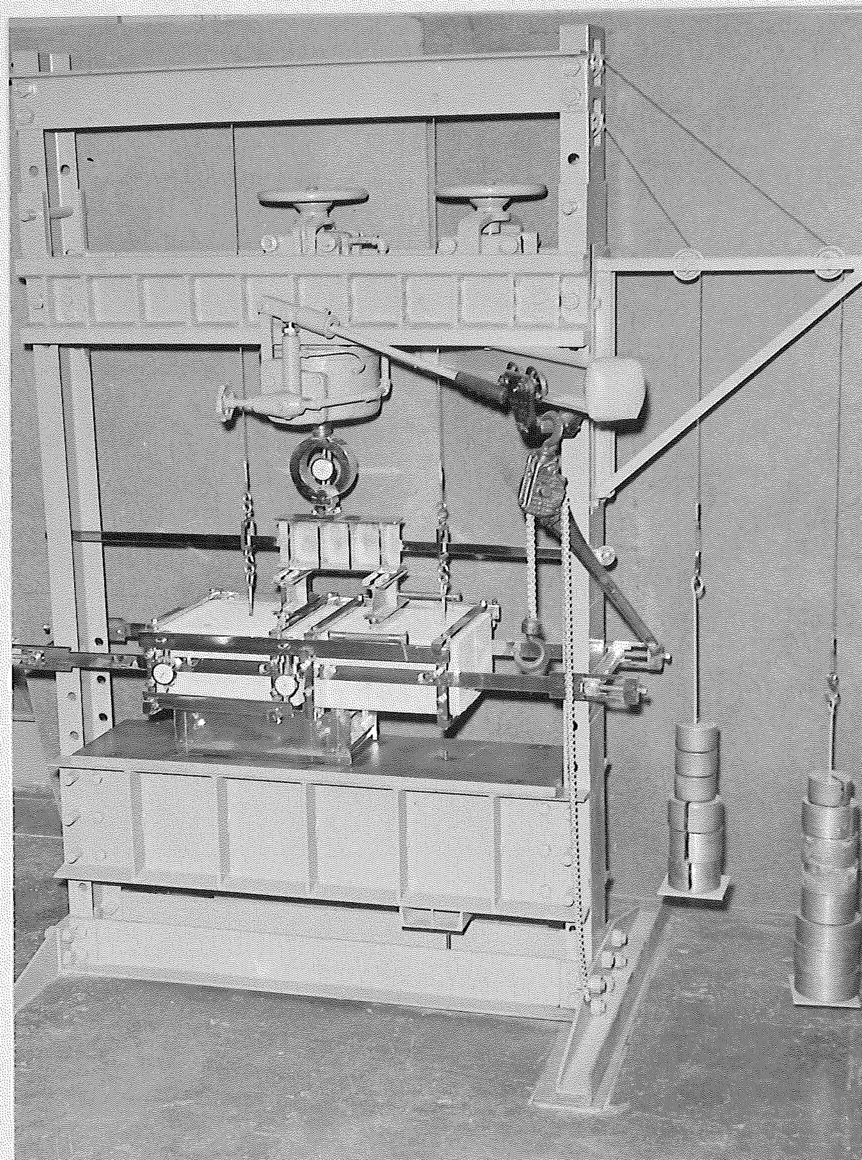
N=Not in specifications or not required. D=Dowel specified in size column. A=As on plans. T=Translode. OC=Optional with contractor. SP=When allowed in special provisions. D or G=Joints at 60 ft. have dowels. Those at 30 ft. have reinforcement carried through a grooved or ribbon joint. V=Variable as follows. 21"-19"-20" for lane widths 10'-12'-11'-13' respectively. NS=not specified.

*Based on 1941 "Summary of Concrete Road Specifications" by Portland Cement Association.

Evaluation of Load Transfer Devices. A few years ago the Department, realizing the importance of the joint design problem, made a preliminary investigation of several prominent load transfer devices with the view of developing a method for evaluating joint units in terms of load transfer ability, mechanical stability and their relative affect upon stress distribution in the concrete surrounding the joint units. After developing an appropriate theory, test procedure, and the equipment shown in Figure 24, and after performing tests on a few joint units, the investigation had to be temporarily suspended because of lack of personnel due to the war. Earlier studies by the Department concerning the theory of load transfer have been mentioned previously in the report.

The joint units which have been thoroughly tested have been evaluated in Table XXI on the basis of their relative spacing in the joint so that the stress in the concrete surrounding the unit would not exceed the assumed design value of 375 pounds per square inch. The relative ultimate shearing strength and total deflection of the various joint units are also included in Table XXI Joint unit LRC (1-1/4" x 15" dowel with rotor sleeve) with the greatest permissible spacing of 24 inches was taken as 100 percent for the purpose of comparison.

The data presented in Table XXI although not conclusive, discloses several significant facts which may be helpful in establishing specification requirements for joint units. In the first place, the evaluation brings out the importance of joint unit spacing in relation to its performance. In other words, from an economic standpoint, the more efficient and perhaps more expensive joint units may be spaced at greater distances between centers along the transverse joint and still give equal or



Equipment and Loading Device used for
Testing Load Transfer Units (39 F-1 - 18)

Figure 24

TABLE XXI

TENTATIVE EVALUATION OF LOAD TRANSFER
UNITS ON WHICH TESTS HAVE BEEN COMPLETED

Symbol	Ultimate Shear Strength of unit V-in lbs.	Tested to Max. repeated Shear of V-lbs.	Relative Deflection in 10^{-4} inches at V=1000 lbs.	When total combined stress in concrete at edge due to load, warping and impact does not exceed 375 p.s.i. for h = 7" and k = 100 p.c.i.		
				Max. Shear V - lbs.	Joint Spacing in inches (a)	Rating %
LRC-0	17,000	3000	17.5	2930	24	100
CC-0	7,500	2250	23.1	2470	20	83
1-EFS-0	9,000	2000	37.7	1850	16	65
N-0	7,500	1000	50.0	1400	12	50
GRC-0	12,850	2500	63.0	1190	11	46
I-0	11,500	1500	75.0	980	11	40
B-0	7,000	1250	81.1	895	9	37
2-EE-0	10,000	2000	181.0	125	2	8

$$\text{Rating} = \frac{a}{24} \times 100$$

Load transfer units were installed in concrete test specimens. 7" deep, 12" wide and 15 inches long for each half of joint. Concrete designed for 3500 pounds compressive strength, Modulus of Elasticity of 5,000,000 p.s.i.

better performance than the cheaper and less efficient units which tests indicate must be spaced at very close intervals. Second, the data indicate that the common 3/4" x 15" dowel should be spaced at intervals of less than 12 inches for efficient performance. Third, for maximum performance in load transfer ability with low deflection values, it will be necessary to resort to special and approved structural members, or round bars with metal sleeves to uniformly distribute the load stresses in the surrounding concrete, or to proprietary devices of high efficiency.

The New Jersey Highway Department (22) after studying the joint design problem from a practical standpoint in the field, has come to the conclusion that the most satisfactory method of load transfer for heavy duty pavements consists of a design which tends to develop the highest possible rigidity consistent with practical considerations and cost. To realize this end they have used rectangular steel dowel bars 5/8" x 2" x 20" spaced at 12 inch centers with excellent results during the last 10 years. They propose even heavier units for the post war period. They firmly believe that it is a wise policy to provide an adequate joint structure in order to preserve the investment in the pavement and that the need for such rigid joint design on heavy duty pavements justifies the extra cost.

Requirements for Load Transfer Devices. From the above researches and experiences it is obvious that a satisfactory load transfer unit must be designed and constructed in accordance with certain sound principles.

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(22) Griffen, H. W.: "Transverse Joints in the Design of Heavy Duty Concrete Pavements." Proceedings of the Highway Research Board, 1943.

It is believed that these principles are:

1. That they should be economically justifiable.
2. The units should be simple in design in order that they may be practical to install and permit positive encasement by the concrete.
3. They should not permit high localized stresses in the concrete at the joint face which would ultimately result in plastic flow and failure of the joint as a whole.
4. They should be capable of distributing the load stresses throughout the adjacent concrete in order that such stresses should not exceed the allowable design value.
5. They must permit free movement of the abutting slabs at all times.
6. They must retain their mechanical stability under wheel load frequency comparable to that for which the adjoining slabs were designed.
7. They must be constructed in such a manner as to meet specified performance requirements relative to stress relief and stiffness. Unfortunately, at the present time insufficient factual data is available from which one can establish definite specification requirements.

On the basis of the above principles an attempt will be made to substantiate the recommendations set forth for load transfer devices. The load transfer devices included in the evaluation study mentioned above were found to fall into three distinct classes from the standpoint of stiffness or load-deflection relationship. These classes are referred to as being stiff, medium or weak and designated respectively as Class 1, 2, and 3. See Figure 25.

By means of Westergaard's (23) stress analysis and the theory of load transfer design advanced by Kushing and Fremont (24) it is possible to establish relationships between joint unit spacing and the resulting stresses in the concrete slab, provided the load-deflection characteristics of the particular joint unit or load transfer device are known. Tables XXII and XXIII have been prepared to show the effect of reduction in subgrade support on slab stresses when load transfer devices possessing different degrees of stiffness are considered. In preparing Table XXII reference is made to Table VII, Part I and to Figures 26, 27, 28 and 29. The unit stress values in Table VII, as a matter of convenience, have been recomputed on the basis of "a" = 7 inches instead of 7.8 and averaged to obtain stress values for the condition when k = 200 as employed in the design analysis. The load-deflection curves in Figures 26, 27, 28 and 29 are based on the performance of the various load transfer units under laboratory tests using the equipment shown in Figure 24 and the theory for joint spacing advanced by Kushing and Fremont (24).

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(23) Westergaard, H. M.: "Stresses in Concrete Pavements Computed by Theoretical Analysis". Public Roads, Vol. 7, No. 2, April, 1926.

(24) Kushing, J. W. and Fremont, W. O.: "Design of Load Transfer Joints in Concrete Pavements." Highway Research Board Proceedings, November, 1940.

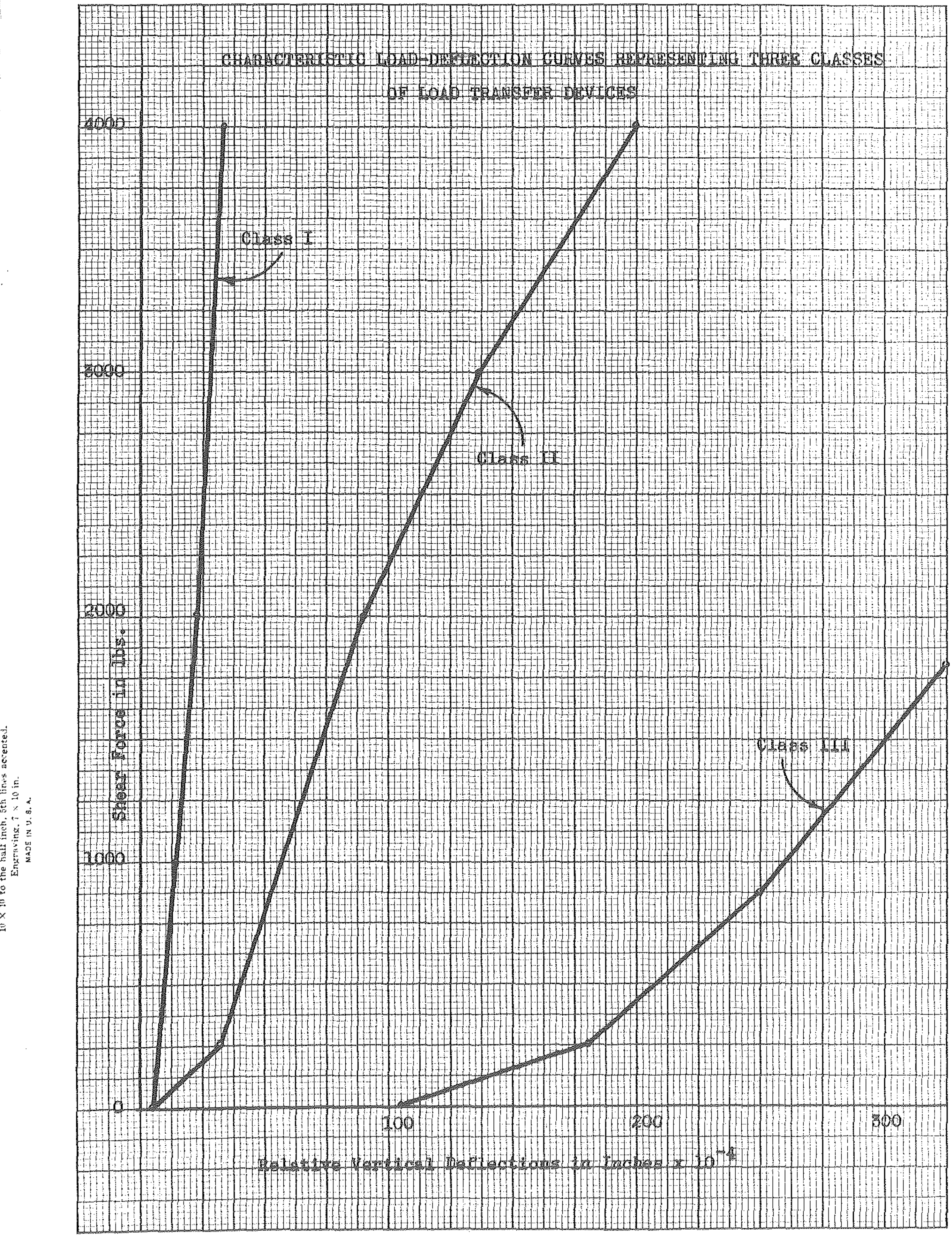
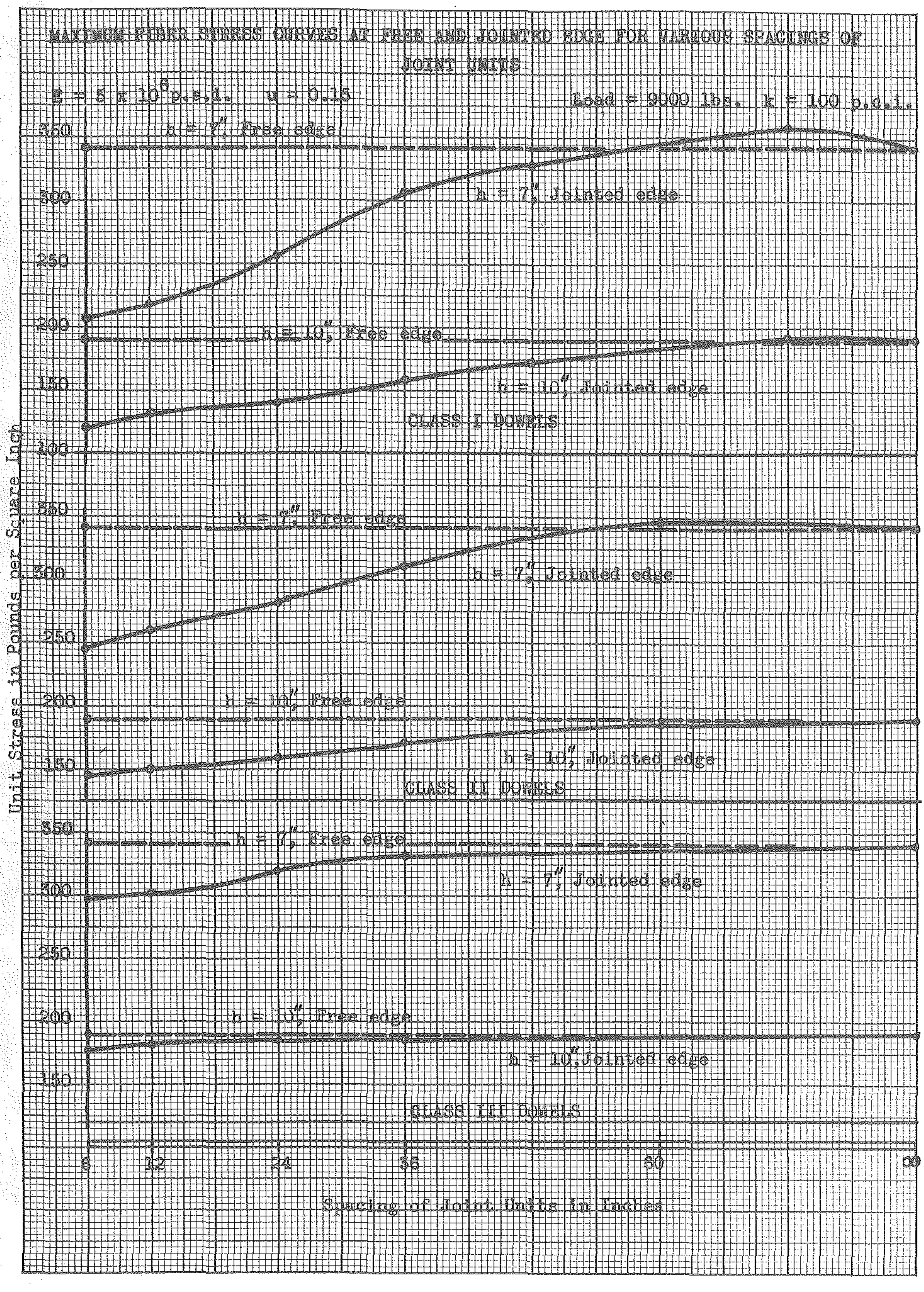
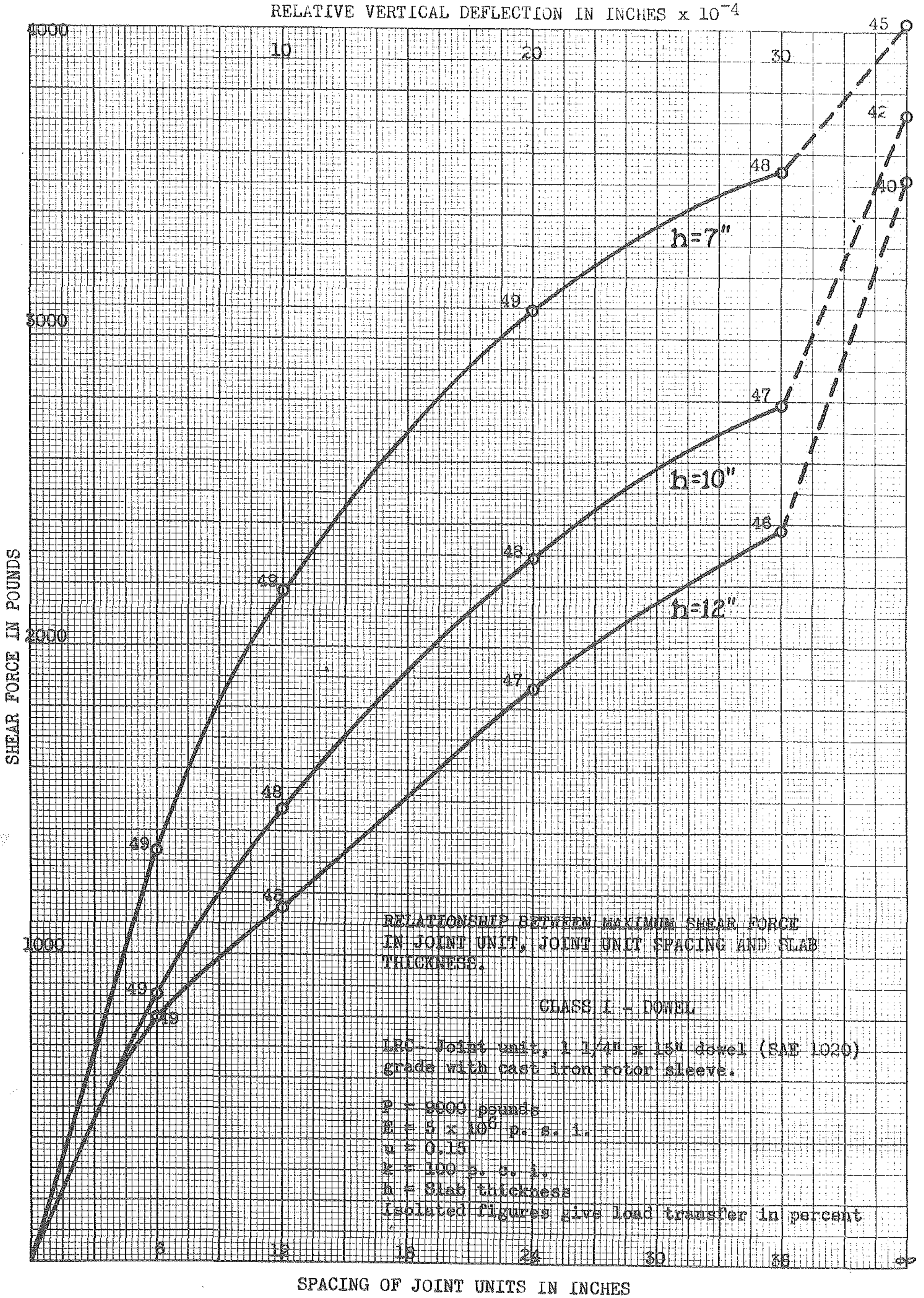

CHARACTERISTIC LOAD-DEFLECTION CURVES REPRESENTING THREE CLASSES
OF LOAD TRANSFER DEVICES
4000
3000
2000
1000
0
Shear Force in lbs.
Class I
Class II
Class III
100
200
300
Relative Vertical Deflections in Inches x 10^-4
KEUFFEL & ESSER CO.
10 X 10 to the half inch, 5th lines accented.
Engraving, 7 x 10 in.
MADE IN U. S. A.

Figure 25



MAXIMUM FIBER STRESS CURVES AT FREE AND JOINTED EDGE FOR VARIOUS SPACINGS OF JOINT UNITS
E = 5 x 10^6 p.s.i.   u = 0.15
Load = 9000 lbs.   k = 100 p.c.i.
h = 7" Free edge
h = 7" Jointed edge
h = 10" Free edge
h = 10" Jointed edge
CLASS I DOWELS
h = 7" Free edge
h = 7" Jointed edge
h = 10" Free edge
h = 10" Jointed edge
CLASS II DOWELS
h = 7" Free edge
h = 7" Jointed edge
h = 10" Free edge
h = 10" Jointed edge
CLASS III DOWELS
350
300
250
200
150
100
6
12
24
36
60
Unit Stress in Pounds per Square Inch
Spacing of Joint Units in Inches

Figure 26



RELATIONSHIP BETWEEN MAXIMUM SHEAR FORCE IN JOINT UNIT, JOINT UNIT SPACING AND SLAB THICKNESS.

CLASS I - DOWEL

LRC- Joint unit, 1 1/4" x 15" dowel (SAE 1020) grade with cast iron rotor sleeve.

P = 9000 pounds
E = 5 x 10^6 p. s. i.
u = 0.15
k = 100 s. c. i.
h = Slab thickness
Isolated figures give load transfer in percent

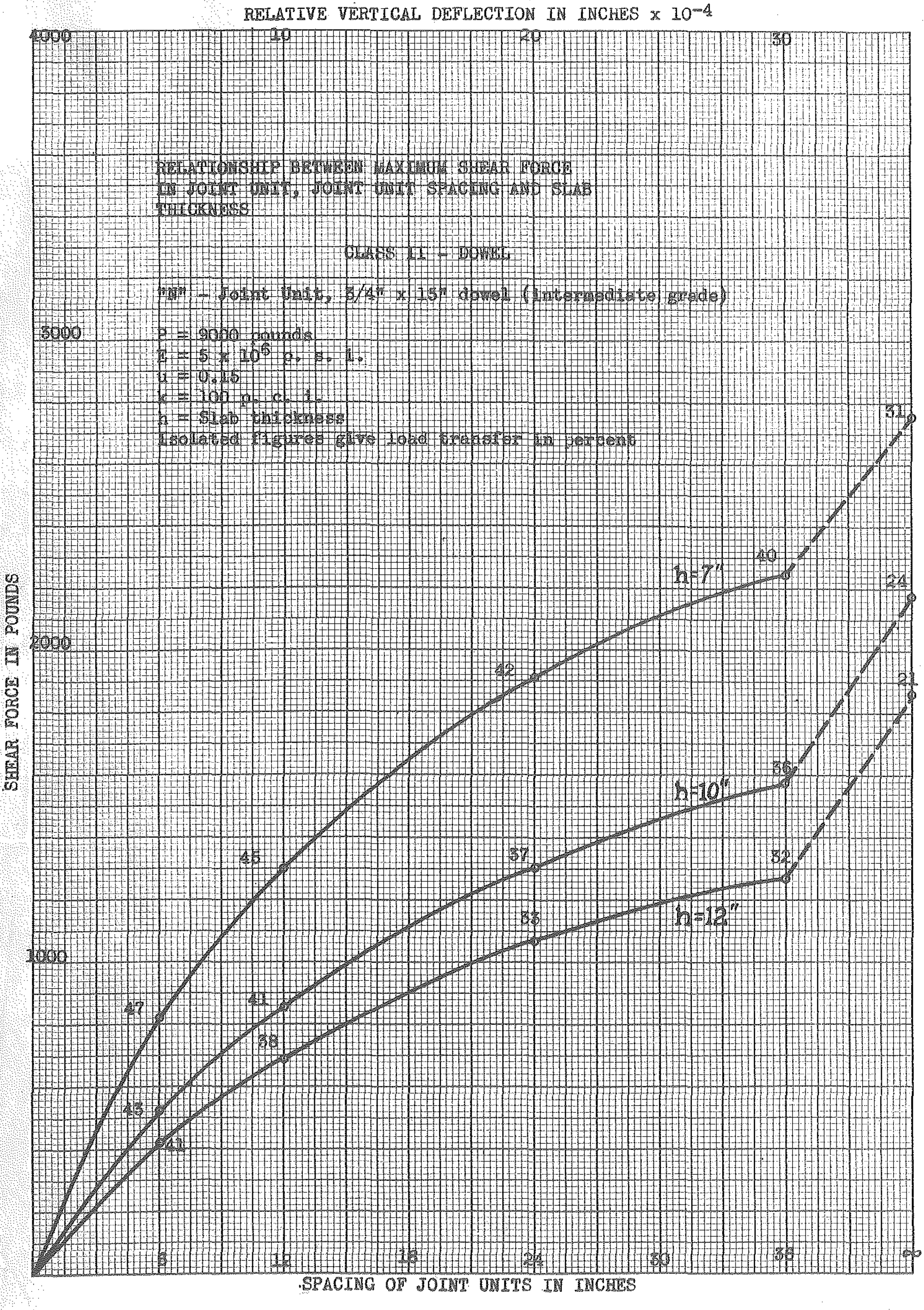

RELATIVE VERTICAL DEFLECTION IN INCHES x 10^-4
RELATIONSHIP BETWEEN MAXIMUM SHEAR FORCE IN JOINT UNIT, JOINT UNIT SPACING AND SLAB THICKNESS
CLASS II - DOWEL
"N" - Joint Unit, 3/4" x 15" dowel (intermediate grade)
P = 9000 pounds
E = 5 x 10^6 p. s. i.
u = 0.15
k = 100 p. c. i.
h = Slab thickness
Isolated figures give load transfer in percent
h=7"
h=10"
h=12"
SHEAR FORCE IN POUNDS
SPACING OF JOINT UNITS IN INCHES
4000
3000
2000
1000
10
20
30
6
12
18
24
30
36
∞
47
45
42
40
31
43
41
37
36
24
41
38
33
32
21

Figure 28

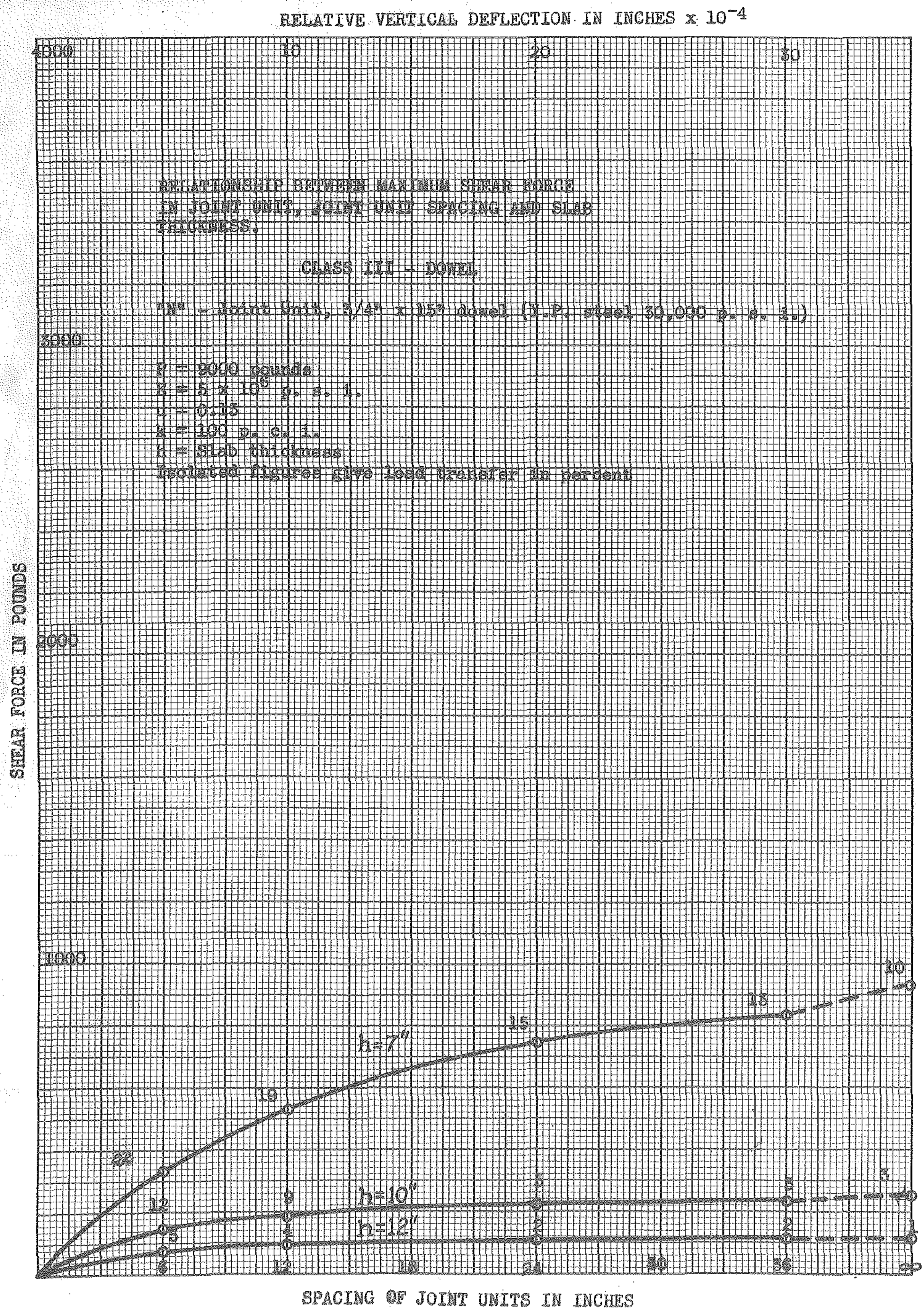

RELATIVE VERTICAL DEFLECTION IN INCHES x 10^{-4}
RELATIONSHIP BETWEEN MAXIMUM SHEAR FORCE IN JOINT UNIT, JOINT UNIT SPACING AND SLAB THICKNESS.
CLASS III - DOWEL
"N" - Joint Unit, 3/4" x 15" dowel (U.P. steel 30,000 p. s. i.)
P = 9000 pounds
E = 5 x 10^6 p. s. i.
u = 0.15
k = 100 p. c. i.
h = Slab thickness
Isolated figures give load transfer in percent
h=7"
h=10"
h=12"
SHEAR FORCE IN POUNDS
SPACING OF JOINT UNITS IN INCHES

Figure 29

TABLE XXII

JOINT EDGE STRESSES VERSUS SUBGRADE MODULUS FOR LOAD OF 9,000 POUNDS AT TRANSVERSE JOINT EDGE

Slab Thickness "h" in inches	Subgrade Modulus "k" p.c.i.	Radius of Load Distribution "a" in inches	Type of Joint Unit See Fig. 25 **	Unit stress in slab for the given joint spacings, "S_e", in p.s.i. 12 inches	24 inches	infinite	Reference
7"	300	7.8 - 7.0*	None	-	-	272-290*	Table VII
7"	100	7.8 - 7.0*	None	-	-	322-343*	Table VII
7"	200	7.0	None	-	-	317	Avg. 290 & 243
7"	100	7.0	Class I	218	255	343	Figure 25
7"	100	7.0	Class II	260	283	343	Figure 25
7"	100	7.0	Class III	307	320	343	Figure 25
10"	300	7.8 - 7.0*	None	-	-	158-167*	Table VII
10"	100	7.8 - 7.0*	None	-	-	182-191*	Table VII
10"	200	7.0	None	-	-	179	Avg. 167 & 191
10"	100	7.0	Class I	126	140	191	Figure 25
10"	100	7.0	Class II	152	160	191	Figure 25
10"	100	7.0	Class II	182	185	191	Figure 25

* - Stress Values in Table VII computed for "a" = 7.8 inches have been revised for "a" = 7 inches.

** Class I = stiff dowel; Class II = medium stiff dowel; Class III = weak dowel.

TABLE XXII

REDUCTION IN STRESSES DUE TO EFFECTIVENESS OF LOAD TRANSFER DEVICE

		CLASS I DOWELS					CLASS II DOWELS					CLASS III DOWELS				
Subgrade Modulus "k" p.c.i.	Slab Thickness "h" in inches	Joint Spacing in inches	Unit Stress in Concrete Se p.s.i.	Impact Factor	Impact Unit Stress	% Stress Reduction	Joint Spacing in inches	Unit Stress in Concrete Se p.s.i.	Impact Factor	Impact Unit Stress	% Stress Reduction	Joint Spacing in inches	Unit Stress in Concrete Se p.s.i.	Impact Factor	Impact Unit Stress	% Stress Reduction
200	7	∞	317	1.50	476	0	∞	317	1.50	476	0	∞	317	1.50	476	0
100	7	∞	343	1.50	514	-8	∞	343	1.50	514	- 8	∞	343	1.50	514	-8
100	7	12	218	1.02	222	+53	12	260	1.09	284	+40	12	307	1.38	424	+11
100	7	24	255	1.02	260	+45	24	283	1.14	322	+32	24	320	1.41	451	+5
200	10	∞	179	1.50	269	0	∞	179	1.50	269	0	∞	179	1.50	269	0
100	10	∞	191	1.50	287	-7	∞	191	1.50	287	-7	∞	191	1.50	287	-7
100	10	12	126	1.04	131	+51	12	152	1.15	175	+35	12	182	1.45	264	+2
100	10	24	140	1.04	146	+46	24	160	1.21	194	+28	24	185	1.47	272	-1

The impact factor considered in the design analysis is defined as:

$$I = \frac{150 - 2f}{100 + f} \quad \text{- XIV}$$

where:

f is the percent load transfer offered by the load transfer device (See Figures 27, 28 and 29).

so that

$I = 1.5$ when $f = 0$

$I = 1.0$ when $f = 50$

The percent load transfer offered by the various devices is represented by the isolated numbers on the curves in Figures 27, 28 and 29. By this method the degree of impact is considered to vary between the values of 1 and 1.5 in relation to the efficiency of the load transfer device.

The data in Table XXIII bring out several significant facts concerning the effect of dowel stiffness in stress relief:

1. That slabs of 7 inch and 10 inch thickness, designed for "k" = 200 p.c.i. without load transfer devices, when placed on a weaker subgrade having k = 100 p.c.i. may have stresses not exceeding those of 375 p.s.i. as considered in the basic design in Part I provided the load transfer features are adequately designed.
2. That the stress reduction due to load transfer, though on the weaker subgrade, is practically independent of the thickness of the slab within the range of thickness from 7 to 10 inches.

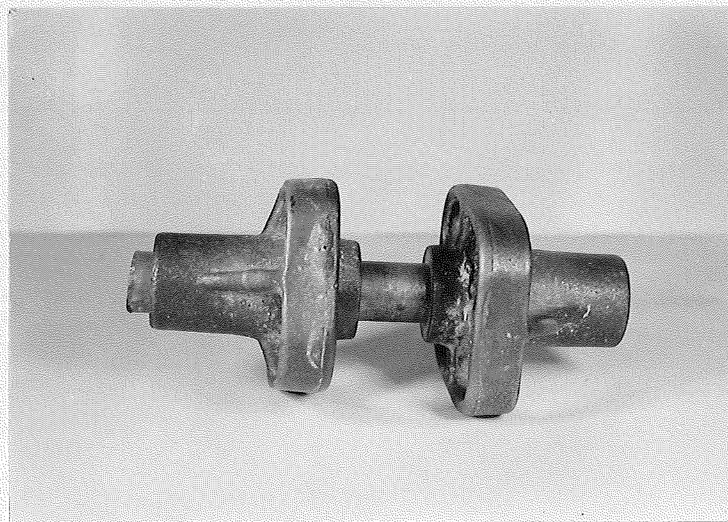
3. Stress reduction affected by load transfer devices under a lowering of subgrade support from 200 p.c.i. to 100 p.c.i. varies from approximately 50% for the best joint unit to practically zero stress reduction for the weaker units. These stress reduction values may even become less as the subgrade is weakened below the value of k = 100 p.c.i. In the future, with careful subgrade preparation, and if the pavement is designed for k = 200 p.c.i. one should not expect a lowering of this value very much below k = 100 p.c.i.

As the above considerations and conclusions have been reached on the basis of free edge stresses which have been computed for different traffic conditions, the traffic effect is included in the computation in Tables XXII and XXIII. It is assumed that the corner stresses will behave in a similar manner.

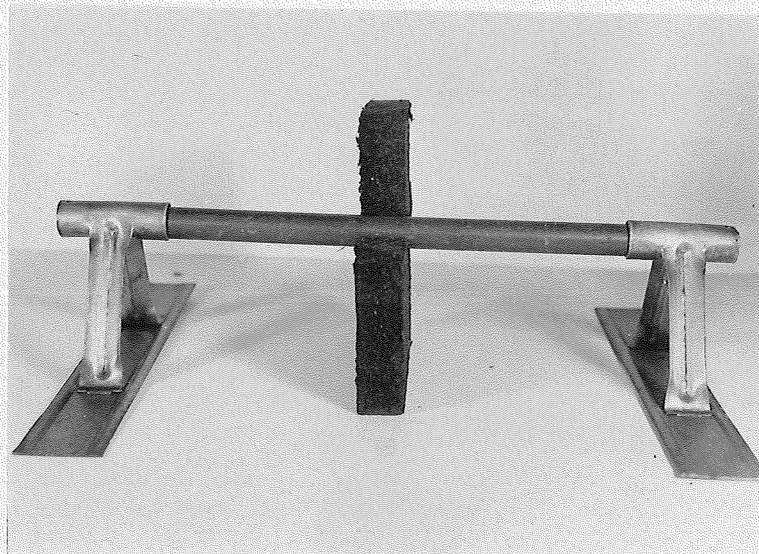
Among the dowel bar types of joint units that have been tested by the Department, there are two units which meet the requirements of the characteristic Class I and Class II load-deflection curves illustrated in Figure 25. They are designated as follows:

1. LRC - 1-1/4" x 15" dowel bar (SAE 1020 grade) with cast iron rotor sleeves for bearing and stress distribution in the concrete. See Figure 30 and Table XXI.
2. N - 3/4" x 15" dowel bar (Intermediate grade) plain. See Figure 30, and Table XXI . In both cases the joint units were tested with a 1 inch joint opening.

When joint units of type LRC were tested to maximum repeated shear of V = 3000#, repeated 25 times, the final deflections did not



A. 1-1/4" x 15" Dowel Bar with Cast Iron Rotor
Sleeve Joint Unit Type "LRC" 39 F-1 (34)



B. Standard 3/4" x 15" Dowel Bar Unit
Joint Unit Type "N". 39 F -1 (29)

Figure 36

LOAD TRANSFER DEVICES. CLASS I AND CLASS II DOWELS exceed those defined by the Class I load-deflection graph in Figure 25 and the strength of the joint units in shear exceeded 16,000 pounds without reaching failure. When tested to a shear force V = 3930 pounds, the maximum permitted stress of 375 p.s.i. in a slab of h = 7" on a subgrade k = 100 p.c.i. is reached when the units are considered to be spaced at 24 inch centers. For a predetermined crack expectancy or on weaker subgrades, this spacing might be too large. Therefore, it is recommended that joint units be spaced at intervals not greater than 15 inches.

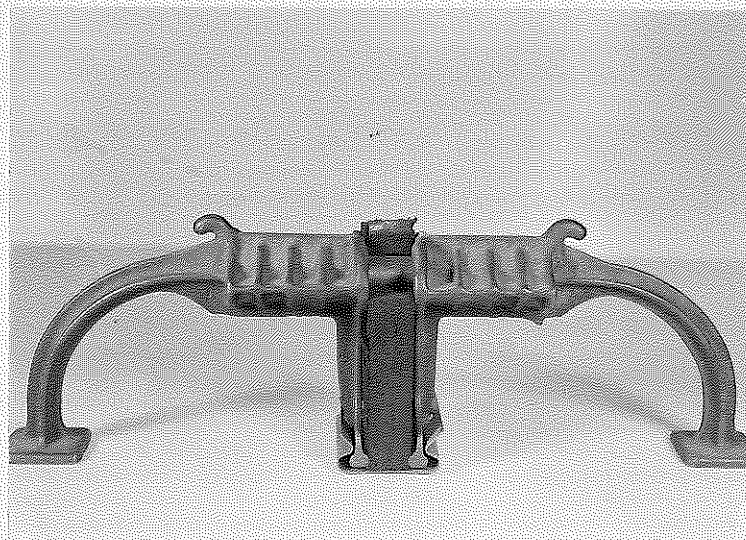
When the joint units of the "N" group were tested under similar conditions, except that a maximum repeated shear of V = 1000 pounds was used, the deflections at the last two applications of load at V = 1000 pounds did not exceed those defined by the Class II load-deflection graph in Figure 25, and the ultimate strength of the joint unit in shear did not exceed 8000 pounds.

The behavior of the "N" type of joint unit is unreliable, as proved by experience, because the specimens were tested only to V = 1000 pounds by repetition, whereas the maximum computed shear is 1430 pounds for a slab stress of 375 p.s.i. at a spacing of 13 inches in a slab 7 inches thick on a k = 100 p.c.i. subgrade. For a given crack expectancy on a weaker subgrade such a spacing might be insufficient for proper slab support.

If the deflection curves for the edge of the slab from corner loads were known, and the value of subgrade modulus "k" from weakened subgrade were available, then it would be possible to design load transfer devices and determine their spacing with greater precision.

Furthermore, in connection with very rigid load transfer units, if the joint units approach absolute rigidity with a 50 percent load transfer ability, they must produce equal stresses in the loaded and unloaded slabs if the load distribution over both adjoining slabs were the same. The wheel load is distributed over a comparatively large area by virtue of the tire, whereas the load transmitted by the joint unit is applied over a very small area. Thus, it is conceivable that the unit stress in the concrete on the load side may be, for example, 90 p.s.i. and in the unloaded slab 210 p.s.i., or approximately 40 percent higher than would prevail under ideal conditions of identical load distribution which would produce a stress of 150 p.s.i. in both slabs. It would appear then, that in the design of all joint units, provision should be made for adequately spreading or distributing the stresses into the mass of the concrete. This necessitates the use of metal sleeves surrounding the dowel bars which can be deeply imbedded and securely anchored in the concrete. This fact has been fully appreciated by various manufacturers of load transfer devices who have advanced various types of metal sleeves. So far the patterns of the proprietary metal sleeves conform very much to the one shown in Figure 31. Laboratory tests indicate that similar types of sleeves are not adequate.

On the basis of the above considerations, to prevent overstressing of the slabs and faulting owing to the weakening of the subgrade from k = 200 to k = 100 p.c.i. and even less, it is believed advisable to use for slabs of from 7 inch to 10 inch thickness the stiffest type of load transfer unit (not less stiff than defined by the Class I curve of Figure 25), of about 50% load transfer (with provision for proper load distribution into the mass of the concrete similar to the distribution which rubber tires produce on the slab surface), and of sufficient length to permit accurate installation.



Illustrating Typical Construction of Metal
Sleeve Common Among Several Makes of Load
Transfer Units. 39 F-1 (27)

Figure 31

This leads us to recommend the use of very stiff load transfer units of the LRC type (See Figure 30), with 1-1/4 inch diameter dowel bars with a high yield point (around 90,000 p.s.i.) and spaced at not more than 15 inches, or some equivalent installation. To prevent deterioration of the metal sleeve it should be properly dimensioned and constructed of metal conforming to A.S.T.M. standards. It may even be advisable to consider slightly larger dowels to compensate for reduction in cross section area due to possible rust effect.

Such a joint installation should effect practically a 50 percent reduction in deflection of slab ends under the same subgrade conditions and reduce the relative deflection of the slab ends to approximately zero.

Sealing of Transverse Joints

In spite of all the care that could possibly be exercised in the design of load transfer units, or in the subsequent construction and installation of the joint assembly in the pavement, there remains one factor in the design and performance of joints which is sadly neglected, and that is the development of ways and means for constructing a transverse joint which is 100 percent impervious to inert foreign matter and especially water.

It is common knowledge that the infiltration of fine gravel particles and soil material into open joints ultimately causes abnormal spalling and longitudinal cracking at joints. It is also known that the passage of surface water to the subgrade causes upward curling action of slab ends at joints which is associated with volume changes of the soil and the formation of ice lenses. Furthermore, a cupping or permanent settlement takes place at joints which is brought about by the presence of free water on the subgrade and the loss of subgrade soil due to pumping of water by repeated deflections of the slabs under heavy loads, or by permanent changes in subgrade support which may occur in the presence of impact and vibration.

The proposition of curing this joint illness cheaply, effectively, and permanetly, is one of the primary problems in joint design, assuming of course that all other factors can be taken care of satisfactorily by proper design and construction methods. There are apparently two possible methods of attacking the problem. They are, first, the construction into the joint of all features capable of excluding foreign matter and water, and second, taking the necessary steps to waterproof and stablize the subbase under concrete pavements either entirely or only in the vicinity of joints. Perhaps it will be necessary ultimately to concentrate all possible efforts on both considerations.

To adequately seal a joint it is necessary to provide means for excluding foreign matter and water from all sides, top, bottom and at both sides. In the case of expansion joints, an attempt is now made to do this by inserting premolded filler materials or wood boards in the joint opening. These materials are supposed to expand and contract in unity with the slab ends and thus keep the joint opening closed. To a certain extent this is true, but much has yet to be done to improve such materials and industry should be capable of providing such a product if the highway industry would strongly demand it.

In regard to sealing the top of expansion joints with bituminous materials, there are two important considerations, (1) the availability of a suitable joint sealing compound and, second, the exercise of proper workmanship in applying it. Only within the last few years have there been available joint sealing compounds which have shown any merit by way of satisfactory performance. These compounds consist in general of a mixture of asphaltic material and rubber. The rubber, in different forms, is combined with the asphalt by various methods. The Michigan State Highway Department has developed specifications for an asphalt-latex joint sealing compound which has been used exclusively on two regular construction projects. After 4 and 5 years in service, without maintenance of any kind, this material still provides a satisfactory seal and seems to possess many more years of useful service.

As to exercising proper workmanship in applying the joint sealing materials, it can be said briefly that present specifications provide for such conditions. In order to obtain positive adhesion of the joint seal material to the concrete, to effect a perfect bond at all times, it will be necessary to provide methods to insure proper preparation of the joint surfaces and in applying the materials. Studies to this effect are now under way in the laboratory.

In regard to contraction joints, no provision is made at the present time to prevent foreign matter and water from entering the joint through the openings which eventually occur at the sides and bottom, or at the surface especially when bituminous premolded parting strips are employed to form the plane of weakness. Off hand, it would seem that the most logical solution to the joint sealing problem would be to surround each joint, at sides and bottom, with a metal shield dimensioned in such a manner as to provide a positive seal without impeding the movement of the abutting slabs. The shield would be constructed and assembled in combinatio with the load transfer assembly. In addition, the joint at the surface would be grooved to proper width and depth and the space formed by the grooving process would be filled with the best available joint sealing compound. In this manner, it might be possible to construct a joint approaching the ideal.

The success of the design covered in this report is predicated on the proper design and construction of the pavement foundation, as well as the construction and sealing of the joints.

CONCLUSIONS AND RECOMMENDATIONS

In the preceding text there have been propounded certain important design features which must be given thoughtful consideration in connection with the construction of future concrete pavements in Michigan. The design requirements mentioned included pavement cross section, its shape and thickness, transverse joints and joint spacing, and joint design. The important subject of pavement foundation, including subbase and subgrade, was intentionally omitted because of the lack of factual data pertaining to soil behavior under concrete pavements.

The recommendations set forth below, and substantiated for the most part by data in the text, are based on sound engineering principles and practice and in keeping with the future trend in concrete construction practice.

Pavement Cross-Section Shape

The uniform, as well as the thickened edge cross section have been considered in the design analysis for comparative purposes. Although a pavement of uniform thickness may cost more to build than one of non-uniform thickness because of slightly increased concrete quantities, it is considered, however, to be a more desirable cross section for the following reasons:

1. The slab thickness can be computed by recognized methods with greater precision and certainty.
2. The trend to wider pavements minimizes the need of the thickened edge.
3. Auxiliary edge strengthening at transverse joints, or at cracks when they occur, is not normally necessary to reduce stresses.

4. Subgrade preparation can be accomplished better and cheaper.

5. It simplifies joint design and construction.

Pavement Cross-Section Thickness

The concrete pavement slab has been designed, on the basis of fatigue, to withstand for a definite period of time the repeated combined critical stresses induced in the pavement by traffic loads, temperature-warping and friction forces. By taking into account the property of fatigue, it is possible to establish a minimum thickness of slab which might be expected to prevent cracking under repeated stresses for a definite period. In the design analysis, slab thicknesses were determined on the basis that, under normal conditions and for the assumptions considered, cracks should not occur within a period of 20 years.

The slab thickness dimensions given by the graph in Figure 10 of Part I, must be considered as a minimum value for any one of the four recognized classes of highways. Although not given in the discussion, there are maximum limits for slab thickness, under the assumptions, which are fixed by warping effect. The recommended minimum slab thicknesses for the conditions as assumed in the analysis and for 100 foot slab lengths are as follows:

		Minimum Thickness
Class I	Highway	9 - 10 inches
Class II	Highway	9 - 10 inches
Class III	Highway	7 - 9 inches
Class IV	Highway	6 - 8 inches

Transverse Joints and Joint Spacing

By virtue of the facts set forth previously in Part II, and because theoretical analysis and factual data are not available for the adequate design of long slabs, it is recommended that continuous slab construction be given consideration in future concrete pavement construction.

Joint Spacing. On the basis of field and research experience, design requirements and assumptions previously discussed, 100 feet is the maximum slab length which should be considered at the present time for continuous reinforced pavement construction.

Contraction Joints. Contraction joints must be constructed every 100 feet in order to create the continuous reinforced 100 foot slab sections. The reinforcement which runs continuously throughout the section is broken at the joints.

Expansion Joints. The use and spacing of expansion joints will be determined by the time of year at which the concrete is poured. September 15th is considered the dividing line between summer and fall construction. It is recommended that during summer construction expansion joints be eliminated except to relieve horizontal pressures at such points as bridge structures, intersections, at sharp horizontal or vertical curves, to minimize the possibility of compression failures, and at other locations required by design or construction progress. During fall construction expansion joints will be installed at intervals of not less than 400 feet, not greater than that of one days pour if warranted by the Engineer.

Reinforcement.

For continuous slab construction it is recommended that the cross section area of the longitudinal steel reinforcement shall be not less than that computed by equation No. XIII, page 43, Part II.

The cross section area of the transverse steel now employed for 22 foot pavements is also sufficient for 24 foot pavement widths.

Joint Design.

It has been pointed out that, in the design of transverse joints, consideration must be given to three important factors, namely: provision for slab movement, sealing of the joint against foreign matter, and to devices for insuring the mutual alignment of the abutting slabs.

Provision for Expansion. In order to reduce horizontal pressures, the following provisions for expansion are recommended:

400 foot sections	1 inch
900 - 1000 foot sections	2 consecutive, 1 inch joints in accordance with Figure 21

Sealing of Joints. In contraction joint construction the grooved and poured type of joint is preferred over that of the premolded bituminous parting strip commonly used. The former method offers a much more positive seal at the top of the joint. However, both methods have the same fault in that they offer no resistance to infiltration at either the sides or the bottom of the joint.

Infiltration at the sides and bottom of joints may be prevented by either of two suggested methods, or more effectively perhaps by a combination of both. If the joint is to be considered only as a contraction joint, then it will be necessary to encase the sides and bottom of the joint in a metal sleeve constructed in conjunction with the joint assembly in such a manner as not to impede the movement of the abutting slabs. If, however, each joint is to be considered an expansion joint, then the employment of a premolded bituminous joint filler material or wood board must be considered. However, experience demonstrates that with perhaps but few exceptions bituminous premolded joint fillers lack the necessary resilience to act in complete unity with the movement of the abutting slabs at all times, and consequently they have not, in the past, been very satisfactory in keeping out inert material or water. The use of pre-compressed wood may overcome this deficiency to a considerable extent, but no data are available to substantiate this fact. Therefore, it may be desirable to consider the use of both principles in joint construction in order to effectively seal each joint.

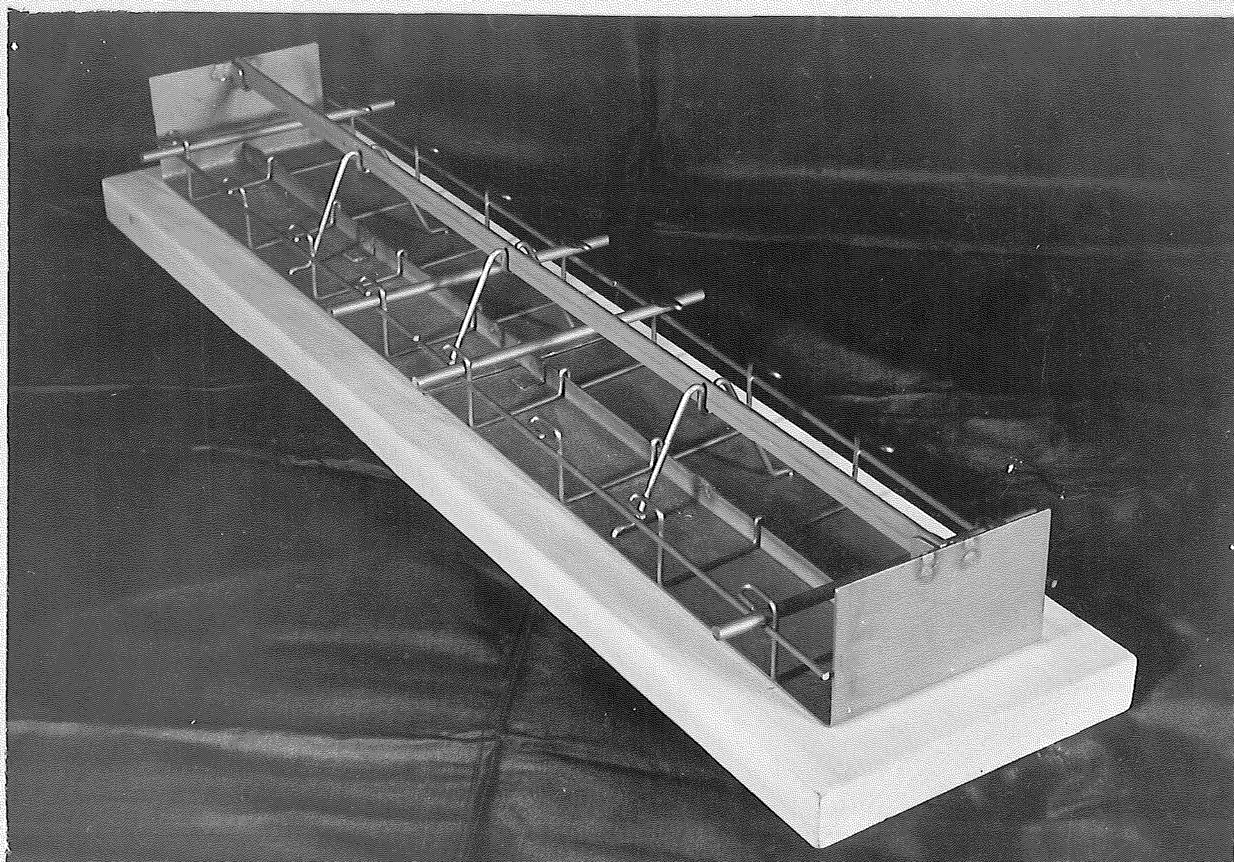
Each type of transverse joint should be sealed only by means of an approved type of asphalt-rubber compound in accordance with Department specifications.

Load Transfer Devices.

It has been pointed out that some kind of mechanism must be provided at joints in order to preserve the riding qualities of the surface and to extend the structural life of the pavement. The ultimate performance to expect from a perfect joint unit device would be a 50 percent reduction in the deflection of abutting slab ends and 100 percent reduction in the relative movement of one slab end to the other. Laboratory tests and field experience indicate that the most rigid type of joint unit is necessary to accomplish this end. Two such devices have been described in the text, namely; 1-1/4" x 15" dowel bars with metal sleeves spaced at 15 inch intervals, and 5/8" x 2" rectangular bars at 12 inch centers employed by New Jersey. Also special treatment of the subgrade at joints has been proposed in conjunction with joint units.

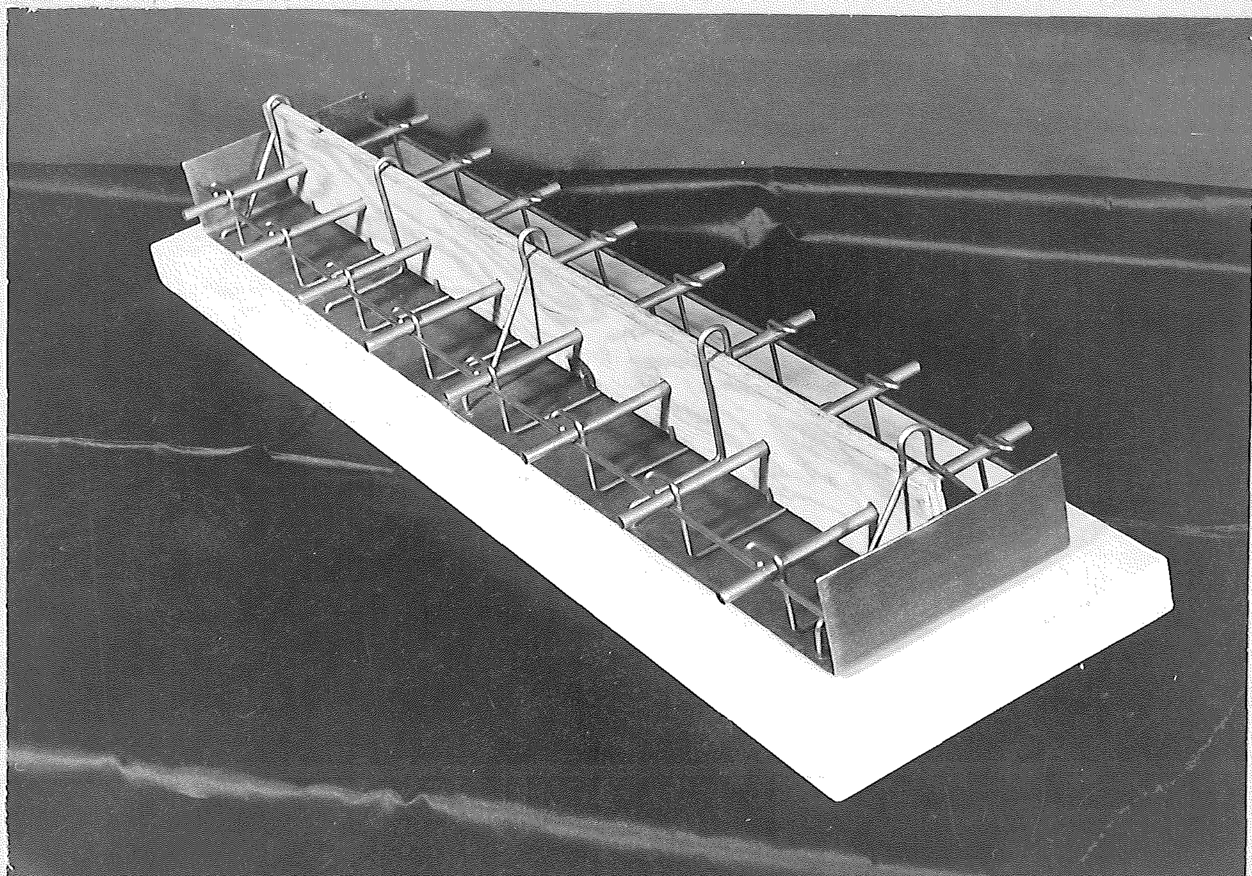
In view of the fact that there is insufficient factual data available upon which one can conclusively base design recommendations as to the proper size, type and spacing for joint units, and since we are fully aware that the present practice of using 3/4 inch dowels at 15 inch centers is entirely inadequate for heavy duty pavements, it is recommended, as a preliminary step towards better joint construction that we change from the present practice of using 3/4" x 15" dowels at 15" centers to that of 1-1/4" dowels at 12" centers, until such time that more definite data can be obtained on the performance of different designs of load transfer units.

In regard to the use of metal sleeves in conjunction with dowel bar joint units, tests and experience substantiate the fact that they are a desirable feature in joint unit construction, since they tend to preserve the original efficiency of the joint unit, and will also effect the distribution of stresses throughout the adjacent concrete. To this end the metal sleeves must be so designed and constructed as to provide for adequate and uniform stress distribution, comparable to the distribution of surface loads by tires and in that respect it is imperative that they be firmly anchored in the concrete.



VIEW OF MODEL ILLUSTRATING PROPOSED DESIGN OF
CONTRACTION JOINTS

Figure 32



VIEW OF MODEL ILLUSTRATING PROPOSED DESIGN OF
EXPANSION JOINTS.

Figure 33

TABLE XXIV

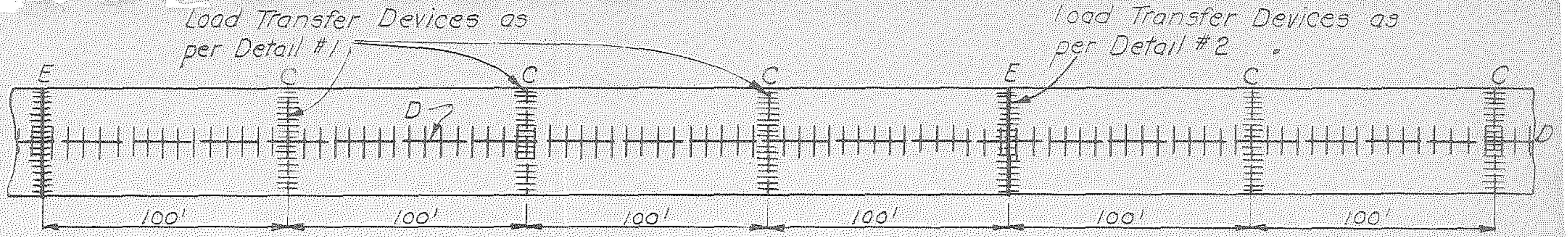
SUMMARY OF DESIGN RECOMMENDATIONS

for

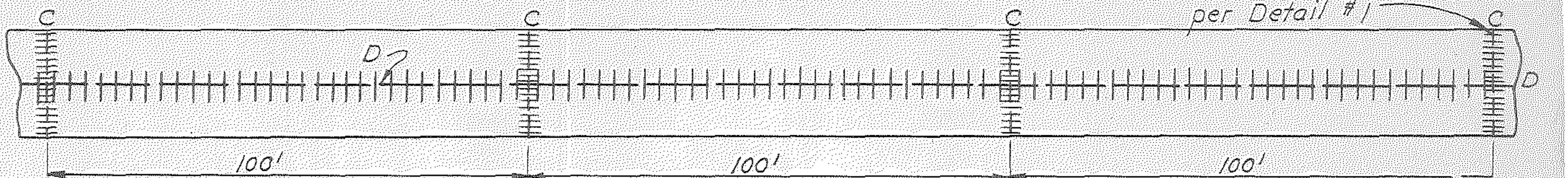
CONTINUOUS UNIFORM REINFORCED CONCRETE PAVEMENT

Highway Class	Slab Thickness (inches)	Reinforcement lbs/100 sq. ft. (approx.)	Joint Spacing: Expansion	Joint Spacing: Contraction	Joint Design*: Load Transfer	Joint Design*: Filler	Joint Design*: Seal
I Expressway	9-10	98	No Expansion Joints during summer season except at structures, intersections and locations specified. During fall construction, commencing Sept. 15, expansion joints will be spaced at intervals of not less than 400 feet or not greater than that of one days pour as directed by the Engineer	Spaced at 100 foot intervals.	1-1/4"x15" Dowel Bars spaced at 12 inch centers for both Expansion and Contraction Joints.	1" wood board at expansion joints. Contraction joints formed by groove 1/2" wide and 2" deep. Metal parting strip 1" high placed at bottom	Asphalt-latex joint seal compound to be used at expansion and contraction joints
II Heavy Primary	9-10	98					
III Light Primary	7-9	85					
IV Secondary	6-8	78					

*A galvanized metal shield will be required at the bottom and edges of joints to prevent infiltration of foreign matter. It will be fabricated in conjunction with the dowel bar assembly.



FOR USE BETWEEN SEPTEMBER 15 AND APRIL 15



FOR USE BETWEEN APRIL 15 AND SEPTEMBER 15

SECTION OF 22' CONCRETE PAVEMENT

JOINT LEGEND

E — 1" Transverse Expansion Joint with Load Transfer Devices

D — Longitudinal Contraction Joint with Tie Bars

C — Transverse Contraction Joint with Load Transfer Devices

Note Spacing of Expansion Joints during Fall Construction may, at the judgement of the Engineer be increased to and not exceed the length of a days pour.

MICHIGAN STATE HIGHWAY DEPARTMENT
CHARLES M. ZIEGLER
STATE HIGHWAY COMMISSIONER

SKETCH SHOWING

SUGGESTED JOINT SPACING AND TYPE FOR POST WAR PROJECTS

APRIL 16, 1945

Revised Aug. 9 1945 F-3-A-116