Technical Supplement

to

Highway Use and Highway Costs

A Report

of the

JOINT STATE GOVERNMENT COMMISSION

to the

GENERAL ASSEMBLY

of the

COMMONWEALTH OF PENNSYLVANIA

Session of 1953

The Joint State Government Commission was created by Act of 1937, July 1, P. L. 2460, as amended 1939, June 26, P. L. 1084; 1943, March 8, P. L. 13, as a continuing agency for the development of facts and recommendations on all phases of government for the use of the General Assembly.

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LETTER OF TRANSMITTAL

To the Members of the General Assembly of the Commonwealth of Pennsylvania:

The Joint State Government Commission presents herewith a technical supplement to its report on the use of the Commonwealth's highways and the costs attributable to vehicles of different types. The study was undertaken by the Commission under authority of the Act of 1939, June 26, P. L. 1084, Section 2(e), upon the suggestion of The Honorable John S. Fine, Governor of Pennsylvania, that the effects of trucks upon highways be ascertained.

The investigations and analyses were made under the immediate supervision of the Commission's Executive Committee. The cost approach directed by the Executive Committee differs from past allocations of highway expenditures in that actual, rather than hypothetical, costs, and all, rather than selected, areas of costs are considered. Though the relative cost positions of different vehicles have been established, cost differentials have not been calculated.

The Commission acknowledges the cooperation of the Highway Research Board, the U. S. Bureau of Public Roads, the American Association of State Highway Officials, the Pennsylvania Department of Highways, and the Pennsylvania Department of Revenue, who furnished certain data, and of the Pennsylvania State Police, who assisted in the conduct of weight and use surveys.

BAKER ROYER, *Chairman*

Joint State Government Commission
Capitol Building
Harrisburg, Pennsylvania

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INTRODUCTION

Highway transportation generates both private and public costs. Private costs—purchase price of vehicle, operating expenses, time in transit, inconvenience—are met directly by vehicle owners and operators. In Pennsylvania, public costs—for highway construction, reconstruction, maintenance, and administration—of the state highway system are recouped by means of highway-user charges.

Changes in traffic volume and traffic composition have their first impact upon private, rather than public, costs. Changes in private costs to highway users are generated by changes in both highway utilization and vehicle characteristics. For example, increases in transit times for specific vehicles are occasioned by increases in numbers of similar vehicles and by changes in numbers and characteristics of dissimilar vehicles. Other private costs, such as insurance charges, are similarly affected.

Increases in private costs eventuate into demands for new highway facilities and preservation, within practical limits, of existing roads. In the conversion of private costs to public costs, the extent of demands for highway service determines in large part whether existing highways are preserved or new highways constructed.

For predicted traffic volumes and compositions, highway facilities are constructed in the light of contemporary design and technology. Private costs, and the conversion of private costs to public costs, arise because predictions are not precise and design and technology change. Current estimates, current designs, and current technology determine today's public costs, which are prorated among contemporary highway users.

Three classes of highway costs are assignable to motor vehicles:

1. Costs, generated by use, of preserving (within practical limits) the carrying capacity of existing roads
2. Costs of new construction and reconstruction
3. Administrative costs.

Engineering analyses of vehicle-highway relationships were undertaken with a view of providing bases for allocation of the first and second of these cost groups, which represent the major portion of total highway costs.

For the purpose of these studies, costs of existing highways are those of preserving design carrying capacities of the road surfaces, in terms of changing traffic volumes and compositions. Costs of preserving carrying capacities of existing highways take into account the critical factors of numbers and weights of axles but do not take into account vehicle dimension and performance characteristics.[1]

Vehicle-highway cost relationships may be determined by statistical analyses where precise highway design and certain utilization data are not available, and by engineering analyses where detailed cost, design, and utilization data are available.

Engineering analyses of existing highways were made. The objectives of these analyses were: (1) the examination of available information for the proration, to vehicles utilizing the highways, of costs of preserving carrying capacities of existing highways, and (2) the testing of highway design systems against performance of actual highways. For the first of these, sufficient information concerning traffic, soils, construction histories and costs, and maintenance costs is not available. These

[1] Costs arising from gradients and alignments inadequate for present-day traffic are prorated to groups of vehicles or vehicle users only when actual costs of correcting these inadequacies are incurred; they are then considered as costs of new construction or reconstruction.

types of information for specific sections of highways are not normally compiled in Pennsylvania. However, analyses of existing highways did provide a basis for testing highway design systems.

Costs of new construction and reconstruction of highway facilities for anticipated numbers and types of vehicles take into account engineering relationships between numbers and weights of axles and highway cross-sections, as well as engineering relationships between the dimension and performance characteristics of vehicles and required gradients, curvatures, and numbers of lanes of highways. The first of these relationships, involving numbers and weights of axles (of critical importance in the actual wearing-out of highways), necessitates review and compilation of systems of highway design. The second, involving dimensions and performance (of importance in the obsolescence of highways), is the subject of series of highway geometric design standards.

Detailed data not presently available are required for precise engineering solutions of these problems. However, on the basis of available data, the following conclusions may be drawn:

1. Satisfactory highway pavements, both rigid and flexible, may be designed and constructed for all classes of traffic. The economic limit of highway loading is not established directly by highway design considerations.
2. Accurate traffic data, including volume and loading characteristics, are necessary for adequate design of highway pavements. The most promising design methods appear to be those based on equivalent wheel-load repetitions.
3. Accurate measurement of subgrade supporting power, under the worst conditions encountered in actual use, and proper treatment of subgrade to develop maximum support are required for optimum pavement construction.
4. Loss of subgrade support can be prevented by effective permanent drainage. Proper drainage design requires detailed study of soil characteristics, geology, and topography.
5. Subgrade conditions in Pennsylvania generally require a minimum thickness of twelve inches of special subgrade under rigid pavements on Class 1 and Class 2 highways. To prevent "pumping," the top six inches of special subgrade should be of special gradation and should be mechanically stabilized.
6. Present thickness design of rigid pavements in Pennsylvania is satisfactory. Thicknesses greater than those now in use may result in shorter pavement life unless slab lengths are reduced.
7. Pavement life, for both rigid and flexible pavements, may be increased materially by the use of wider lanes. Increased width minimizes edge-loading conditions and "tracking."
8. Current secondary design details—load transfer devices, reinforcement, expansion and contraction joints, joint spacing, etc.—are satisfactory for present axle loadings where adequate subgrade support is provided. Changes in axle loadings might require modification of these designs.
9. Construction standards and specifications require constant review and study. Simplification of standards by limitation of the number of special designs is desirable.

Section I

EXISTING HIGHWAYS

Highways are designed for specified total numbers of vehicle passages and specified vehicle and axle-weight distributions. Number of vehicle passages and vehicle and axle-weight distributions define the life of any highway.

For analytical purposes, existing highways may be utilized to:

1. Test highway designs by checking estimates of highway life, based on design procedures, against observations of life of actual highways
2. Estimate the costs (in dollars or in man hours and materials) of preserving (within practical limits) the carrying capacities of existing roads, if actual traffic patterns conform to those predicted. (Estimation of preservation costs is necessary if the cost of existing highways is to be prorated among different types of vehicles.)

For any specified distribution of axle loads, the life of a highway is dependent upon the thickness and strength of the pavement, base course, and subbase course, and upon the support provided by the soil beneath the highway.

Two general types of pavements, flexible and rigid, are of importance in Pennsylvania. A flexible pavement consists of a bituminous wearing surface over a stabilized aggregate base. It has been demonstrated by field tests that such pavements may deflect up to 0.2 inch without cracking. (B-17)[1] On the other hand, rigid pavements, composed of Portland cement concrete, may crack when the deflections under load are in excess of 0.05 inch. Portland cement concrete pavements are relatively brittle. Tests indicate that, while Portland cement concrete pavement can undergo millions of minute bendings which develop less than one-half the ultimate stress, increases in magnitudes of deflections cause increasingly rapid deterioration, to the point where a single deflection of a critical magnitude will cause complete failure of the pavement. Flexible pavements also show long life when subjected to minute deflections resulting from low loads, but under large loads develop ruts and displacements.

The rigid pavement, which acts as a beam in spanning a weak foundation, differs markedly in principle from the flexible pavement, which acts as an impervious carpet over a load-supporting subgrade. However, the life of both types of roads is directly dependent on the amount of deflection under wheel load. The principle that degree of pavement deflection depends primarily on quality of subgrade support has long been recognized from observations that certain types of subgrade soils, especially when dry, could carry safely any axle loads imposed, without the addition of paving of any type. The addition of a pavement may make a good subgrade into a good highway, but the same treatment will by no means convert a poor subgrade into a good highway.

Subgrade support is the strength or resistance offered by the underlying soil against the downward or outward push of wheel loads. This strength can be measured directly either by a field shear test or by a laboratory triaxial compression test. In the latter, the compressive stress necessary to fracture a soil sample is measured under conditions where the soil is restrained laterally to simulate natural support.

The subgrade support can also be measured indirectly, in the field, by determining the force necessary to push a circular plate a certain dis-

[1] Numbers in parentheses refer to bibliography listings, page 93.

tance into the ground. When the plate is 3 square inches in area and the unit load at 0.1 inch settlement is compared to 1,000 (the empirical standard stress denoting very good subgrade support), the test is known as the California Bearing Ratio test. When larger plates (12, 18, or 30 inches in diameter) are used, the test is called a plate loading test. In this procedure, the plate corresponds to a wheel, and the measurements of the stress necessary to produce certain settlements are noted. From these tests, it is possible to estimate the settlement of subgrade soil under any actual tire load on any type or thickness of pavement.

These test methods indicate if the existing foundation soil is of adequate strength to support design loads or if the subgrade should be reinforced with better soil. The methods are not new and have been employed advantageously by many states in providing highway pavements capable of supporting any design loads.

Identical wheel loads on pavements of identical composition and thickness cause different rates of pavement deterioration unless the subgrade support is, in all cases, equal.

In order to make precise predictions of highway life, detailed information is required concerning:

1. Support capacity of subgrade soil
2. Thickness and strength of pavement, base, and subbase
3. Numbers and weights of axles of vehicles which have passed over the highway.

An exploratory study of highway life was made on the basis of 128 sample sections of Pennsylvania roads. The study was based primarily on soil, traffic, and construction data compiled by the Pennsylvania Department of Highways.

For 16 of the sample locations (new loadometer stations), traffic and axle-weight data were available for a three-year period (1949-1952). For 14 additional sample locations (old loadometer stations), traffic data were available regarding the number and type of commercial vehicles, but only limited information was available regarding axle loadings. For the remaining sample points, the total traffic was known from automatic recorders or was estimated from data for nearby stations, and the percentage of commercial vehicles was estimated from experience and observation. However, little precise information was available concerning the number of very heavy vehicles included in the commercial traffic category for each road.

Available information regarding the thickness and type of pavement, base, and subbase was obtained from construction or maintenance records.

Since adequate subgrade support information was not available, plasticity and moisture test data from 16 of the sample locations (new loadometer stations) were used. These tests are but indirect measures of soil support; they are not precise and are too few in number to show variations in soil properties even at these 16 sample locations. At the remaining 112 stations such data were not available.

In the surveys of 16 sample locations in different sections of the state, information was compiled in the form of line diagrams of the highway sections. Drainage conditions, fill and cut depths, and pavement deterioration were noted at the same time soils information was listed. The highway soils classification adopted for this work is:[2]

1. Very good subgrade — compact gravel, crushed stone, and gravel and sand mixtures; also slag fills over 4 feet deep
   k—over 275 C.B.R.—over 20
2. Good subgrade—compact sand, gravel, and clay mixtures not appreciably softened by ground water
   k—225-275 C.B.R.—12-20
3. Fair subgrade—fine sand, sand and silt, and sandy clay mixtures of low compressibility
   k—175-225 C.B.R.—7.5-12

[2] *k* = modulus of subgrade reaction (psi/in); C.B.R. = California. Bearing Ratio (percent). A list of symbols appears on page 91.

4. Poor subgrade—densely compacted clay and silt of low compressibility, or partially saturated clays well above water table
   k—100-175 C.B.R.—3-7.5
5. Very poor subgrade—plastic clays and clayey silts, moderately to highly compressible, and saturated silts
   k—below 100 C.B.R.—below 3

While the number of survey points is certainly not adequate to establish complete soils data for Pennsylvania, available information indicates that most subgrade soils in the state are not of high supporting power. In general, the subgrade seems to be of the third, or fair, category. In many areas, most of the subgrade soil is of the fourth class, and in a few areas most is of the fifth class. In very few cases are soils with high supporting power encountered for significant distances. For the state as a whole, it appears that treatments of subgrades to develop additional supporting power should be studied. In areas where subgrade soils are poor, life of new pavements might be increased considerably by application of methods of subgrade treatment.

With pavements of various types, ages, and uncertain construction histories included in the relatively small number of sample points studied, it was impossible to obtain satisfactory comparisons of pavement performance under various subgrade support conditions. Information for the 16 points studied is summarized in Table 1. Pavement deterioration under moderate traffic has not been extensive except where subgrade support has been poor. Cases of unusual pavement damage seem to

Table 1

HIGHWAY DESIGN AND PERFORMANCE AT SIXTEEN SELECTED LOADOMETER STATIONS

	Traffic Data		*Pavement Design*				*Pavement Performance*			
Loadometer Station Number	*Average Daily Traffic (1951)*	*Percent Commercial*	*Type**	*Thickness (Inches)*	*Subgrade Soil Classification†*	*Drainage*	*Age (Years)*	*Relative Amount of Cracking*	*Estimated Remaining Life (Years)*	*Relative Maintenance Cost*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*	*(11)*
12.......	4,850	22%	Pcc	9″	4	Poor	10	Low	8	Very low
53.......	3,800	19	Pcc	9-7-9″	2 and 3	Fair	15	Very low	3	Very low
59.......	2,750	17	Pcc	9-7-9″	3	Poor	23	High	4	High
88.......	3,200	18	{Pcc B	{6-8-6″ 2¼″	3	Poor	{29 3	Moderate	4	Moderate
100.......	7,750	27	{Pcc B	{10-8-10″ 1″	2	Fair–Good	{20 11	Low	3.5	Low
104.......	7,150	18	{Pcc B	{10-8-10″ 2″	3	Fair	28	Moderate	12	Moderate
110.......	7,700	28	Pcc	9″	3 and 4	Fair	7	Moderate	6	Very low
123.......	10,700	33	Pcc	10-8-10″	3	Good	17	Moderate	0	Very high
132.......	7,300	21	Pcc	9″	5 and 4	Good	7	Very high	1	Low
136.......	15,900	23	{Pcc B	{10-8-10″ 2″	4	Good	17	Low	7	Moderate
169.......	4,950	27	Pcc	9″	5 and 4	Good	10	Moderate	0 to 5	Very low
182.......	15,400	26	{Pcc B	{10-8-10″ 4″	3 and 2	Fair	{11 2	Very low	0 to 6	Very low
192.......	8,500	17	Pcc	9″	3	Fair	16	Very low	6	Very low
202.......	5,500	28	Pcc	9-7-9″	3 and 4	Poor	21	High	0	Low
461.......	8,000	20	Pcc	10-8-10″	3	Poor	21	——	5	——
503.......	5,200	16	Pcc	9″	3	Poor	3	Very low	12	Very low

*Pcc—Portland cement concrete
B—Bituminous
†See above, pages 4 and 5.

be limited to locations where the subgrade soil or drainage is poor, or where a combination of these factors reduces subgrade support. Conclusions from this study are:

1. Subgrade conditions vary widely within relatively short distances on Pennsylvania highways.
2. Soils generally encountered in highway construction in Pennsylvania do not have high supporting power.
3. Detailed study of subgrade treatment to secure high supporting power is needed to obtain economies in construction.

In view of the foregoing, life predictions made in this study must be considered as first approximations, subject to refinement and revision as more accurate soil and axle-loading information becomes available.

Available information concerning the 128 sample sections was not sufficiently detailed to permit assigning to vehicles and classes of vehicles the costs of preserving the carrying capacities of these roads. However, field inspection of 68 of the 128 sample highway locations showed that the predictions of pavement life agreed well with observed life. Considering that over 100 points were compared in each case, it would appear that the procedures developed in this study have a reasonable degree of over-all accuracy.

The systems of highway design developed in this report take into account the number and magnitude of wheel and axle loads, as well as types of pavements and subgrades. With identical pavements on identical subgrades and with identical *numbers* of load applications per day, deterioration rates will differ unless the cumulative effects of all the repetitive deflections from all the wheels and axles are the same. In order to check designs against existing highways, it is therefore important to: (1) estimate numbers of specific axle weights which have passed over each of the highways; (2) translate the pavement deflections resulting from these loads into cumulative pavement deterioration.

No accurate prediction of pavement life can be made without detailed knowledge of the numbers and weights of axle loads which have been operated over the pavement. Such information had been obtained at 16 Pennsylvania loadometer stations during parts of approximately four days per year in 1949, 1950, and 1951. The times had been selected to give the best traffic information in the limited period of observation. The four times of the year selected were approximately three months apart, and in each of these periods vehicles were weighed at three different eight-hour intervals on typical weekdays to constitute one typical day. The trucks were classified in several categories generally related to axle loading or gross weight.[3] Both weights and axle spacings were recorded. Some of these data were compiled in tabular form and are plotted in Chart I, which presents a comparison of axle loads of trucks registered in Pennsylvania and in other states.

While these data indicate the average axle loads for all trucks weighed on all roads, they do not show specifically what loads could be expected on any one road except at the sampling points. Interview card data for one station were reduced into axle loads, and this curve was compared with the state-wide average previously obtained. This compilation for Loadometer Station 110 is presented in Chart II, which also shows seasonal traffic variation.

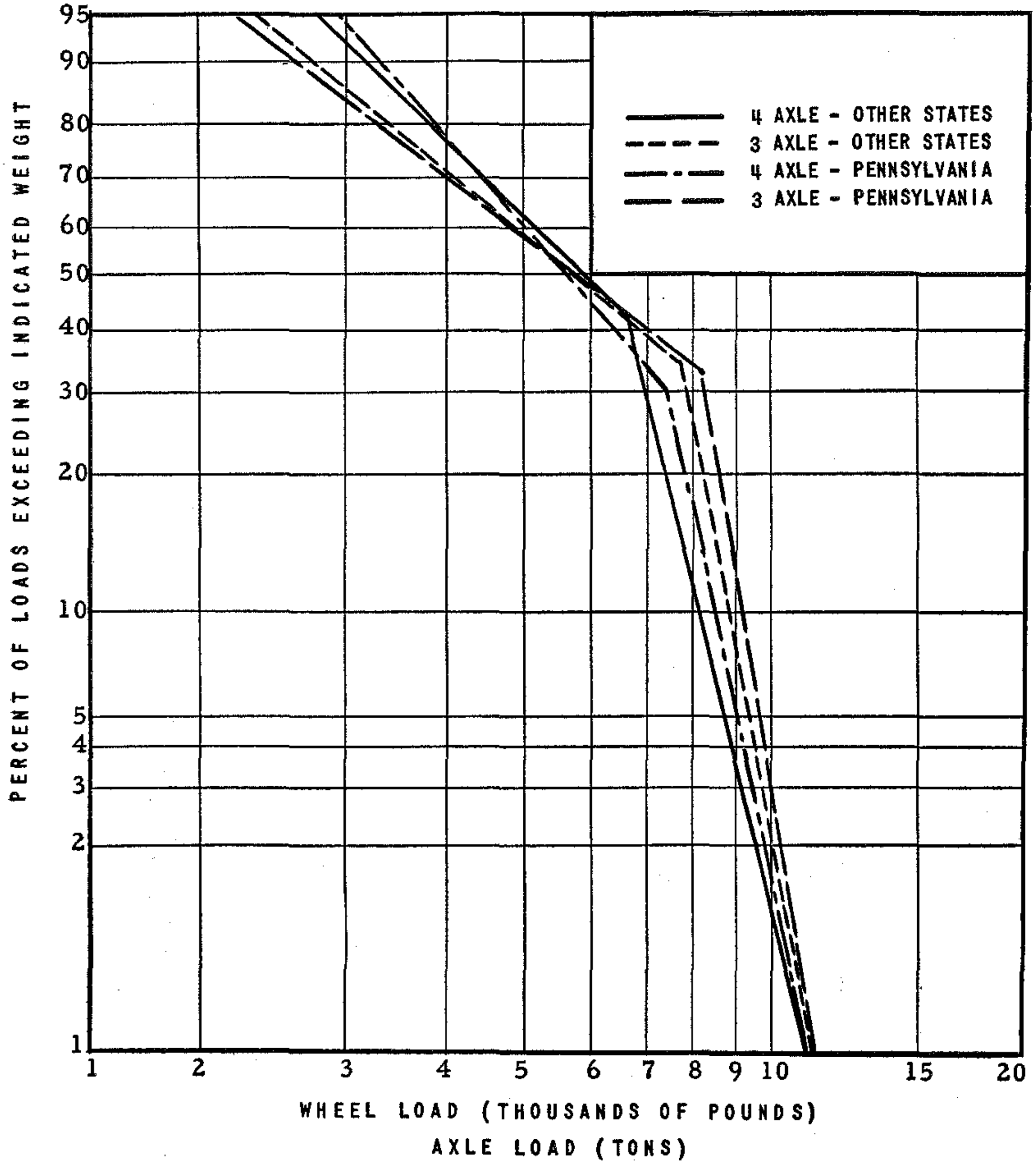
Sufficient variation was indicated between the average and the data for Station 110 to necessitate computation of axle-load distributions for the remaining loadometer stations. This information is shown in equivalent wheel-load concentration factors in Table 2.[4] While these computations produced wheel-load information for the new loadometer stations, only estimates of the total

[3] Single body, 2-, 3-, or 4-axle; truck-tractor semitrailer, 3-, 4-, or 5-axle; truck-trailer, 4-, 5-, or 6-axle.

[4] See Chart VI and Section II.

Chart I

WHEEL AND AXLE LOADS OF TRUCKS REGISTERED IN PENNSYLVANIA AND OTHER STATES*



* 1950 and 1951 data from 14 Pennsylvania loadometer stations.

2

Table 2

Equivalent Wheel Load Concentration Factors for Selected Loadometer Stations: 1949, 1950 and 1951

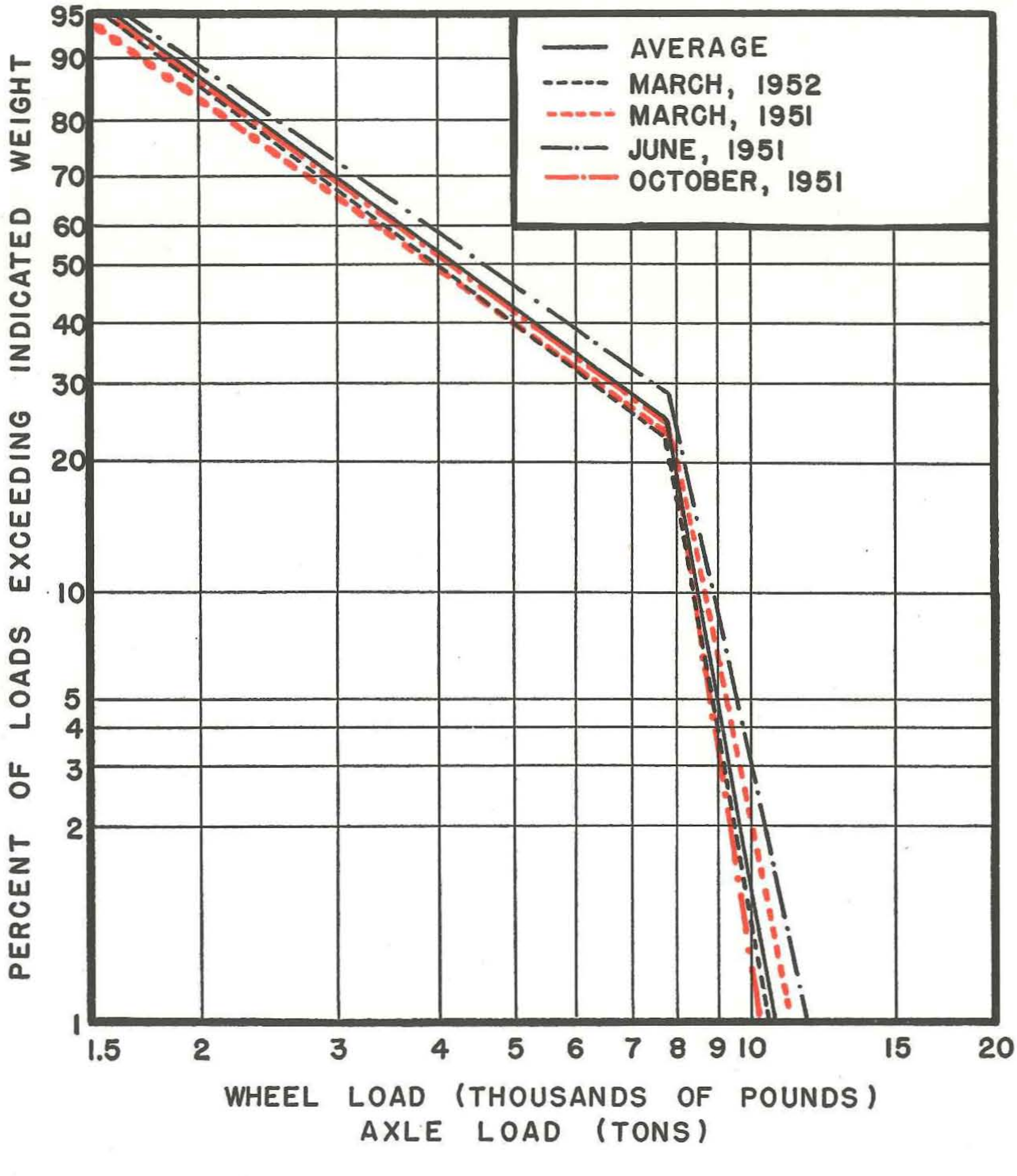
Loadometer Station Number	*Traffic Data*		*Equivalent Wheel Load Concentration Factors**															
	Average Daily Traffic (1949 through 1951)	*Percent Commercial*	*2-Axle Trucks*				*3-Axle Trucks*				*4-Axle Trucks*				*All Trucks*			
			1949	*1950*	*1951*	*Average*	*1949*	*1950*	*1951*	*Average*	*1949*	*1950*	*1951*	*Average*	*1949*	*1950*	*1951*	*Average*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
12	4,100	19	0.95	2.45	1.74	1.71	8.33	5.55	6.10	6.66	3.46	3.45	1.58	2.83	3.10	3.70	3.68	3.49
32	3,300	19	5.75	1.11	...	3.43	7.28	4.02	...	5.65	.21	74.72	...	37.47†	6.27	11.24	...	8.75
53	7,300	22	15.51	7.23	3.47	8.74	7.01	9.08	6.52	7.54	16.53	21.75	2.72	13.67†	12.38	9.08	5.16	8.87
59	3,500	19	2.54	1.66	1.61	1.94	5.54	5.66	7.86	6.35	...	...	0.38	0.38	3.29	2.27	3.63	3.06
88	4,000	28	4.17	9.97	15.48	9.87	4.05	10.57	4.38	6.33	...	...	2.73	2.73	4.11	10.24	10.44	8.26
104	7,900	19	3.50	2.67	2.39	2.85	5.56	6.81	9.46	7.28	2.66	0.79	9.74	4.40	4.41	4.32	7.07	5.27
110	10,100	18	3.04	1.48	2.19	2.24	9.77	10.13	7.83	9.24	2.97	12.39	4.07	6.48	7.61	7.08	5.83	6.84
123	9,100	26	1.50	1.71	2.61	1.94	11.04	7.56	8.97	9.19	3.81	3.32	1.69	2.94	8.47	5.99	7.23	7.23
132	9,000	24	3.03	3.51	3.36	3.30	9.41	7.57	8.08	8.35	2.02	2.88	2.79	2.56	7.75	5.44	5.99	6.39
136	17,000	19	1.83	1.07	2.03	1.64	10.75	8.85	7.22	8.94	4.72	7.46	4.17	5.45	5.92	7.72	5.43	6.36
159	2,600	22	0.76	1.96	1.35	1.35	10.26	4.43	7.15	7.28	0.58	12.03	4.24	5.62	4.28	3.63	4.49	4.13
169	4,950	27	...	8.15	...	8.15	...	4.89	...	4.89	...	1.28	...	1.28	...	6.09	...	6.09
182	20,500	23	6.03	7.09	4.41	5.84	5.53	9.09	9.27	7.96	10.27	5.34	4.68	6.76	6.59	8.27	7.20	7.35
192	10,500	17	2.46	5.06	2.77	3.43	6.71	3.12	3.62	4.48	0.31	4.36	2.76	2.48	4.18	4.08	3.10	3.79
502	10,800	26	5.63	4.18	3.66	4.49	9.65	6.95	9.16	8.59	4.85	8.09	5.30	6.08	7.84	7.08	6.92	7.28

*Concentration factor in equivalent wheel load per truck is obtained by multiplying number of axles in each wheel-load weight range by the corresponding E. W. L. factor and dividing this product by the number of trucks in each category.

†Improbable data; not used in further calculations.

Chart II

SEASONAL VARIATION OF TRUCK WHEEL AND AXLE LOADS: PENNSYLVANIA LOADOMETER STATION NO. 110



number of vehicles and percent of commercial vehicles were available for the remaining stations. It was necessary to estimate for each of the remaining 112 samples an axle-load distribution based on location, volume of traffic, communities served, grades, and bridge restrictions.

Different axle-load distributions obtain in different areas and under different permitted maxima. Examples of load distributions in the state of New Jersey are shown in Charts III and IV. Estimates of general axle-load distributions which would prevail under maxima of 10,000 to 40,000 pounds are shown in Chart V.

The several factors which cause deterioration of a pavement must be evaluated in order to develop a means for relating pavement deterioration to magnitudes of wheel loads. The basic principles involved are easily demonstrated. For example, a sheet of brittle metal may be bent back and forth sharply a number of times to break it. The number of bendings needed to cause failure of the metal depends on the sharpness with which the metal is bent in each of these stress reversals.

This principle, which applies to rigid pavements, was demonstrated by fatigue tests on samples of concrete pavements tested by the Illinois Division of Highways and published in *Engineering Report 34-1*. For concrete pavement slabs, it is known that failure can result from a single loading if a critical maximum deflection results from that load. The magnitude of the stress which causes this failure is called the rupture modulus. The Illinois tests demonstrated that failure can also result from applications of loads smaller than the critical load, provided such loads are repeated a number of times. Theory and findings indicate that these loads and the deflections produced caused cumulative deterioration of the concrete pavement. It was concluded that 75 percent of the rupture modulus stress of a concrete slab could be repeated about 275 times before failure resulted; 70 percent, 1,250 times; 60 percent, 30,000; 55 percent, 160,000; and 50 percent could be repeated millions of times.

From these experiments, a pavement deterioration factor can be related to the ratio between applied load stress and rupture modulus. For example, the 75 percent stress from the above data would produce $\frac{1250}{275}$, or 4.5 times the deterioration of a 70 percent stress. In order to develop this method, it is necessary to determine the fraction of rupture modulus stress which results from each axle load. If the number of axle loads is known and the amount of deterioration occasioned by each axle load can be established, the cumulative pavement deterioration can be determined.

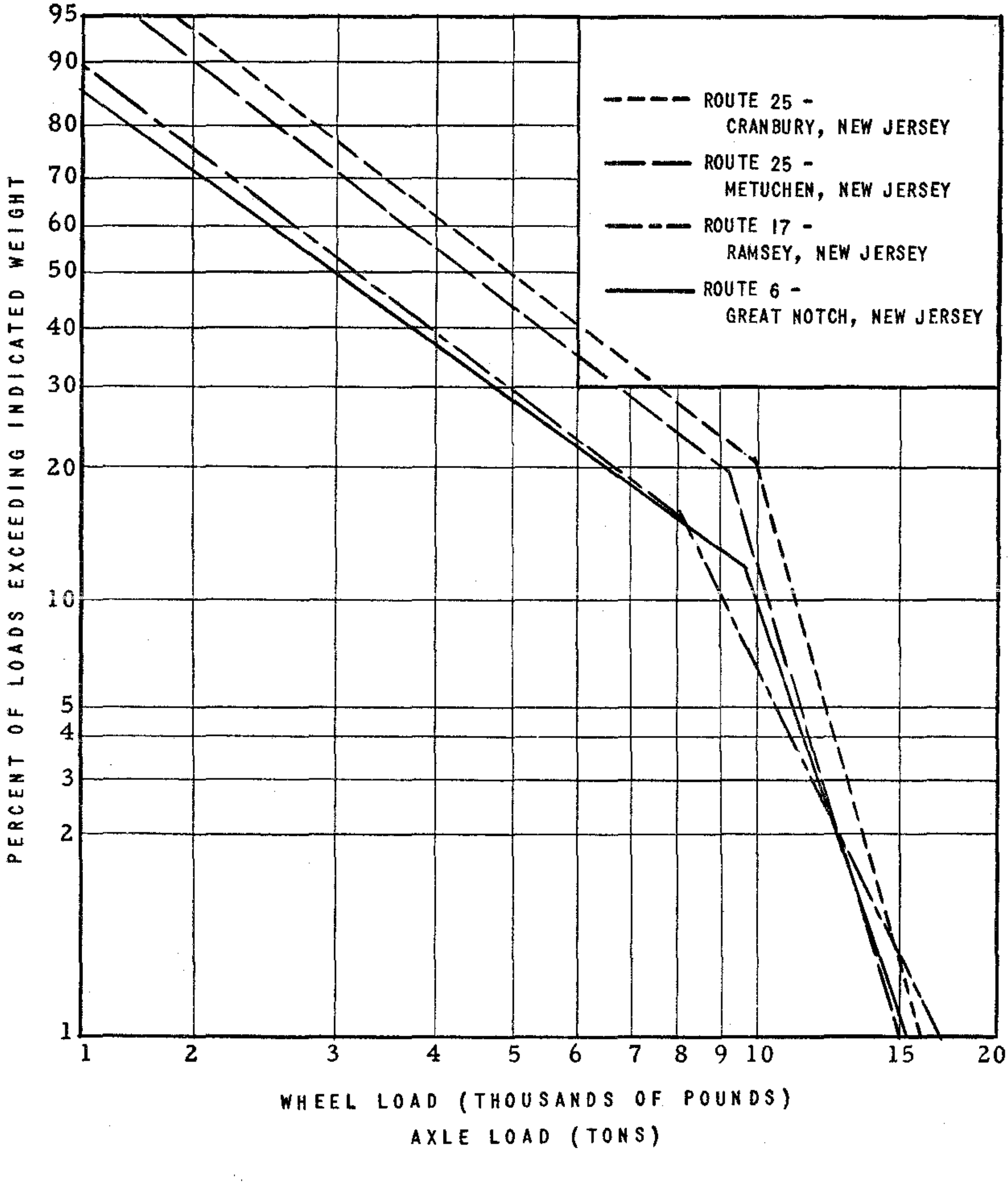
This phase of the problem is treated in greater detail in Section II. It is worthy of note that the stresses in a rigid pavement are due both to wheel loads and to temperature warping of the slab. Under certain adverse conditions, the stress in the pavement may amount to as much as 65 percent of the rupture modulus before addition of wheel-load stresses. It is correct to infer that a section of concrete pavement never subjected to truck loadings would eventually crack and deteriorate due to temperature warping stresses.[5] Wheel-load stresses, when added to high temperature warping stresses, rapidly accelerate pavement deterioration.

The first detailed analysis of this problem was made in 1938 by Bradbury (B-4), who estimated the number of load repetitions a pavement could withstand for wheel loads of 5,000 pounds, 6,000 pounds, and others. The next modification was by the California Department of Highways (B-9), in 1941. It was found advantageous to express deterioration resulting from stresses produced by wheel loads larger than 5,000 pounds as multiples of deterioration caused by a wheel load of 5,000 pounds. These multiples for wheel loadings of 6,000, 7,000, 8,000, 9,000, and 10,000 pounds,

[5] The term "warping stresses" is used synonomously with "restrained warping stresses." There are no true "warping stresses."

Chart III

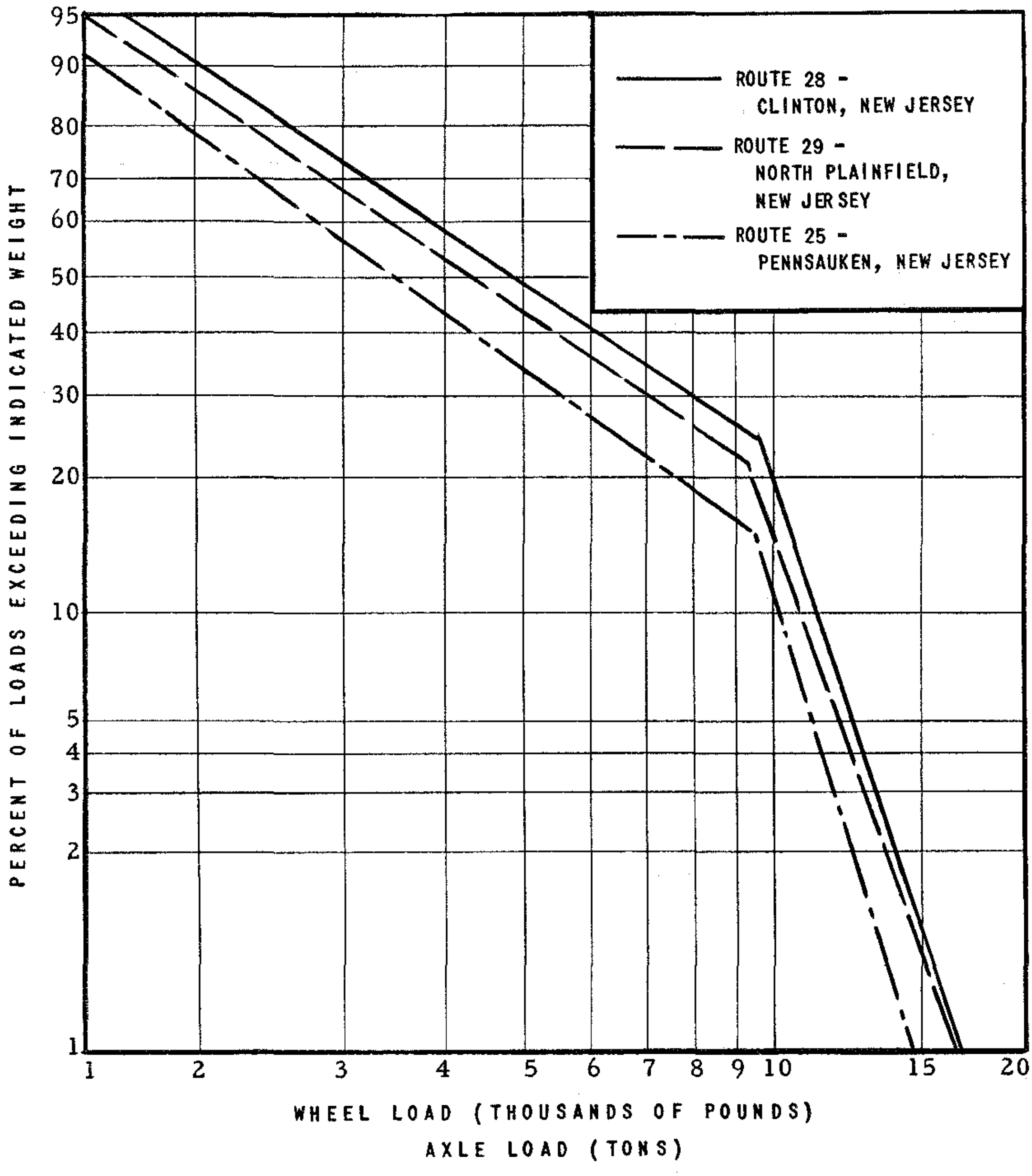
SAMPLE DISTRIBUTIONS OF TRUCK WHEEL AND AXLE LOADS: NEW JERSEY



Source: *Proceedings*, Highway Research Board,. Vol. 28 (1948), p.78.

Chart IV

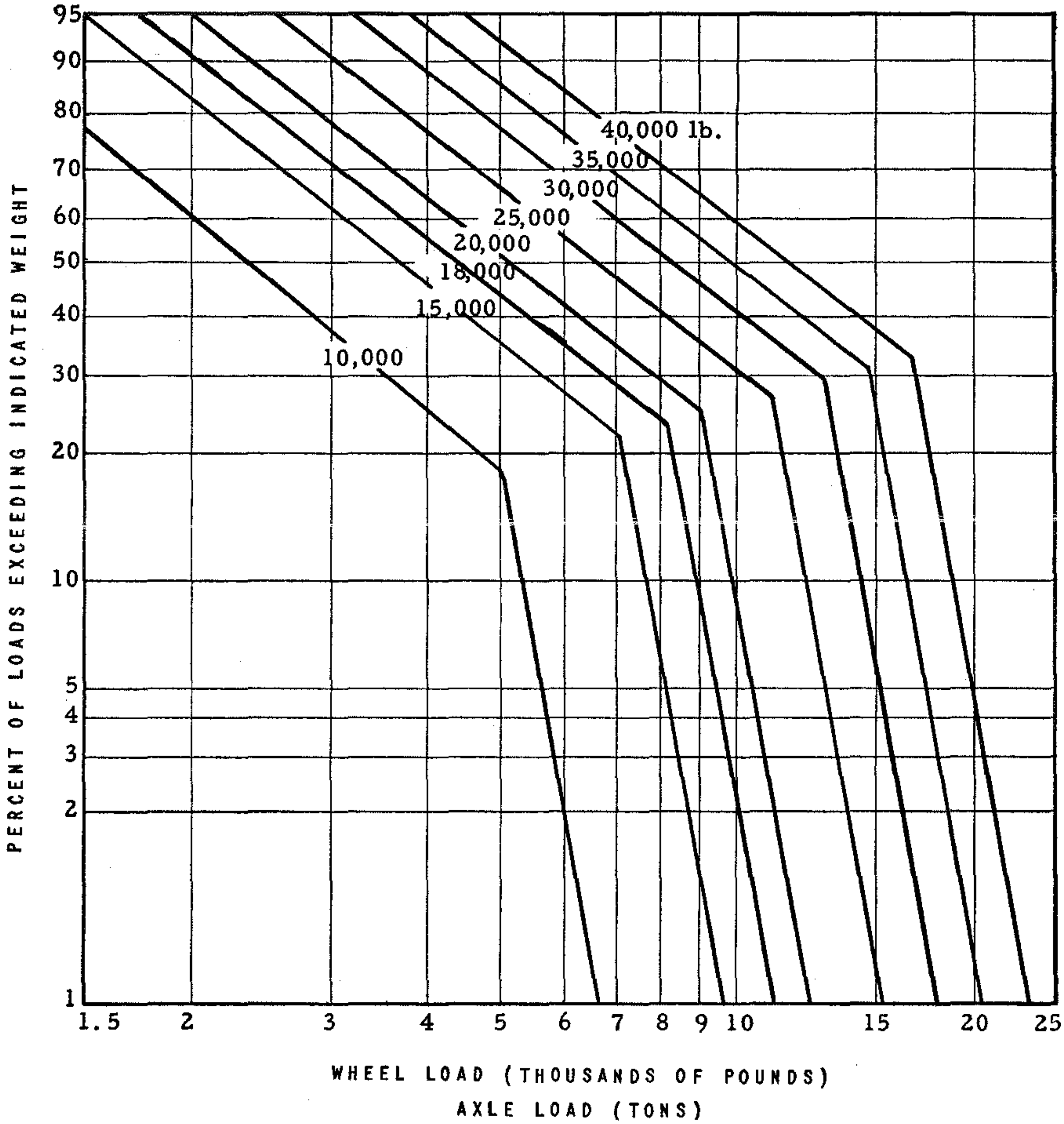
SAMPLE DISTRIBUTIONS OF TRUCK WHEEL AND AXLE LOADS: NEW JERSEY



Source: *Proceedings*, Highway Research Board, Vol. 28 (1948), p. 78.

Chart V

ESTIMATED DISTRIBUTION OF WHEEL AND AXLE LOADS AT PERMITTED AXLE-LOAD LIMITS OF 10,000 TO 40,000 POUNDS



(based upon a factor of 1 for a load of 5,000 pounds) are 2, 4, 8, 16, and 32.

This geometric series has certain inherent advantages: it is a reasonably accurate representation of the true deterioration ratios and is easy to apply in computation. For these reasons, this set of factors has, over the past ten years, been adopted by a number of states as a basis for evaluating pavement deterioration.

Although this specific set of deterioration factors may not be exact, no more accurate series can be developed from present information, and these factors are now widely accepted (B-23). However, before this series was adopted for the present survey, a study was made to determine if a better series could be developed. The basic stress data used in studying this phase were obtained from precise stress computations for 7-, 8-, and 9-inch pavements on subgrades of varying support characteristics.[6] These stress data were then related to a 5,000-pound wheel load, representing unit deterioration. These deterioration factors are shown in Chart VI. From these data, it appears that use of the 1:2:4:8:16 geometric series is reasonable and represents a fairly satisfactory average of all factors, within the range of pavement thicknesses and subgrade supports most frequently encountered in Pennsylvania.

A procedure was developed for predicting the probable distribution of axle loadings for several axle-load limits above and below those of Pennsylvania, and the distributions are shown in Chart V. The data shown in Table 3, derived therefrom, are the estimated percent of axle loads in each weight category and under each axle-load limit.

If the number of axles in each weight category is multiplied by the corresponding equivalent wheel-load (E.W.L.) deterioration factor, the resulting product represents, numerically, cumulative deterioration. If the number of truck axles were the same under modified loadings, the ratio of these products would represent the relative magnitude of pavement deterioration under the modified loads. If a 20,000-pound axle loading (the present legal maximum) is used as a unit basis for comparison, the ratio of cumulative deterioration will represent the relative rapidity of pavement deterioration under other maxima. Chart VII presents comparisons of pavement deterioration at different maximum permitted axle loads.

A procedure for relating pavement deterioration to thickness of pavement and quality of subgrade, and taking into account numbers and weights of axles, has been developed. The proposed design curve was checked against Pennsylvania experience, using the procedures outlined below. For the pavements in the 128 sample sections, the following data were used:

1. Thicknesses and types of pavement, base, and subbase
2. Soil subgrade support[7]
3. Date of pavement construction
4. Present condition of pavement and estimated number of years before replacement would be required.

Using these field data, equivalent wheel-load repetitions were computed from the average daily traffic, the years of highway use, and the axle-load distribution obtained from loadometer surveys. The total equivalent wheel-load repetitions to the estimated fatigue points were computed, using the subgrade and thickness data. From these totals to fatigue points, the estimated numbers of equivalent repetitions to date were subtracted; the difference was the estimated number which could be sustained in the future. Since the rate of application of equivalent wheel loads under present and future traffic as shown by Chart VIII could be esti-

[6] Assumptions of temperature differential, rupture modulus, and fatigue curve, together with computation procedures, appear in Section II.

[7] Since district engineers had no facilities for making either C.B.R. or plate loading tests, the following procedure was employed: The soil characteristics of five major groups, ranging from very good to very poor, were used as bases for classification of the soils actually present at the sample sections.

mated, the remaining life in years could be predicted.

These predictions were checked by comparing the life in years, computed by the above method, with the field life predicted by district engineers (see Chart IX). The composite prediction for the highway samples was sufficiently close to the 45° line representing perfect correlation to make modification of the proposed design curve unnecessary. However, predictions for certain of the 128 sample points may be too high or too low. Considering the accuracy of present field data, it is not possible to state whether such departures are due to faulty data or represent deficiencies in the design curves.

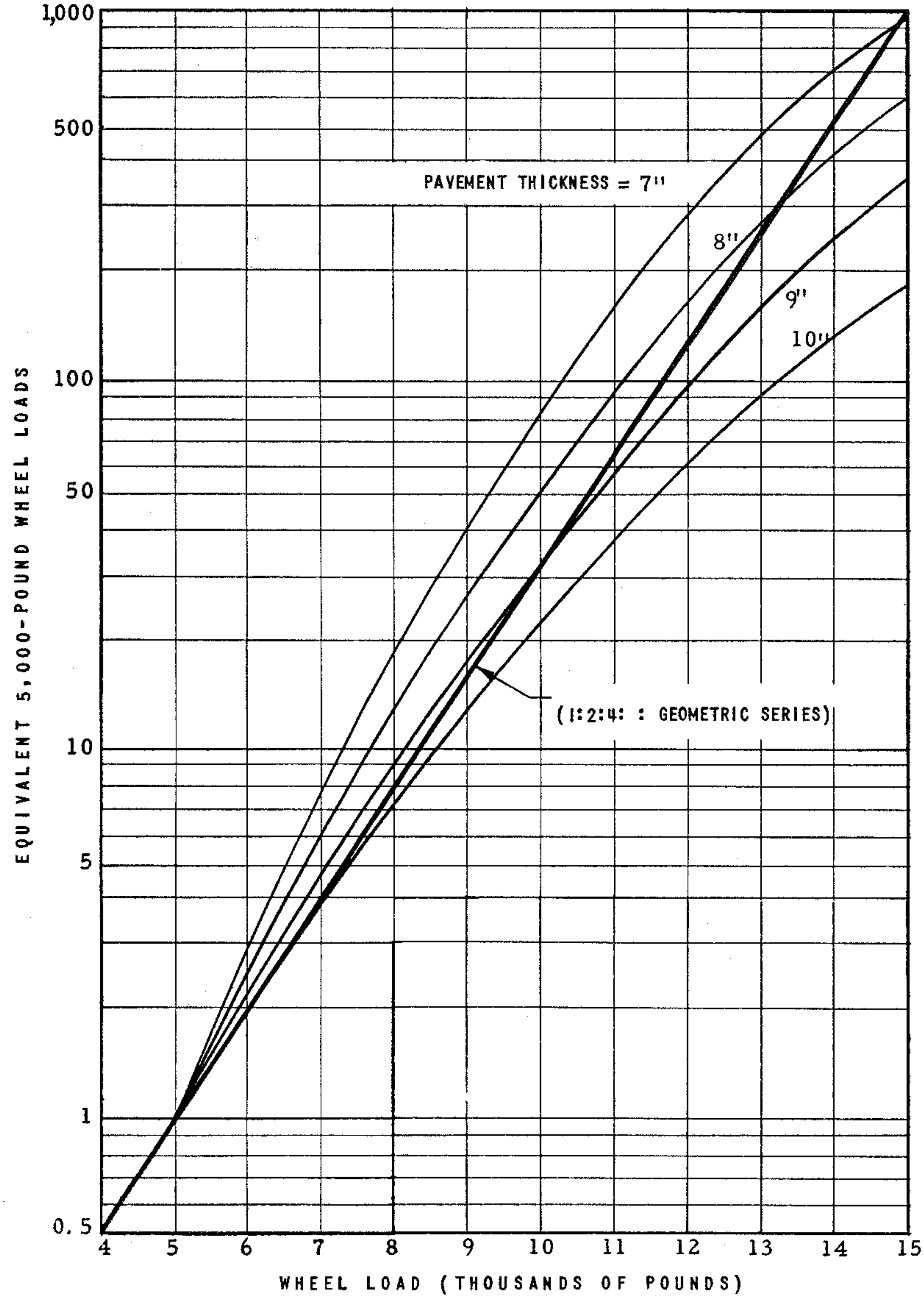
Two points must be kept in mind in considering the design curves presented in Section II: They are intended for estimation purposes only and should not be used for design without more de-

Table 3
ESTIMATED PERCENT DISTRIBUTIONS OF TRUCK AXLE LOADS FOR SELECTED MAXIMUM AXLE-LOAD LIMITS

Wheel Load (000 lbs.)	*Equivalent Wheel Load Factor (Base: 5,000 lbs.)*	*Estimated Percent of Total Truck Wheel Loads for Different Maximum Axle-Load Limits*						
		10,000 lbs.	*15,000 lbs.*	*20,000 lbs.*	*25,000 lbs.*	*30,000 lbs.*	*35,000 lbs.*	*40,000 lbs.*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Less than 4.5	...	79%	60%	43%	30%	17%	10%	5%
4.5 to 5.5	1	15	10	11	10	11	10	6
5.5 to 6.5	2	4.8	7	9	10	9	8	9
6.5 to 7.5	4	1.2	11	6	7	9	8	7
7.5 to 8.5	8	...	10	4	6	6	6	6
8.5 to 9.5	16	...	1	11	5	6	7	7
9.5 to 10.5	32	...	1	11.5	4	4	5	6
10.5 to 11.5	64	...	...	2.9	13	5	4	4
11.5 to 12.5	128	...	...	1.6	9	3	4	4
12.5 to 13.5	256	...	...	...	4.1	12	4	4
13.5 to 14.5	512	...	...	...	0.7	10	3	3
14.5 to 15.5	1,024	...	...	...	1.2	5	12	3
15.5 to 16.5	2,048	...	...	...	...	1.3	9	3
16.5 to 17.5	4,096	...	...	...	...	0.6	6	13
17.5 to 18.5	8,192	...	...	...	...	1.1	2	9
18.5 to 19.5	16,384	...	...	...	...	...	0.7	6
19.5 to 20.5	32,768	...	...	...	...	...	1.3	2.4
20.5 to 21.5	65,536	...	...	...	...	...	...	1
21.5 to 22.5	131,072	...	...	...	...	...	...	0.3
22.5 to 23.5	262,144	...	...	...	...	...	...	0.6
23.5 to 24.5	524,288	...	...	...	...	...	...	0.7
Total Products [Total (E. W. L. factor • %)]		29.4	196.0	1,019.4	4,934.8	28,484.2	129,453.2	946,016.8
Intensity Ratio $\left(\frac{\text{Total}}{\text{Total Col. 5}}\right)$		0.03	0.19	1.00	4.84	27.94	126.99	928.01

Chart VI

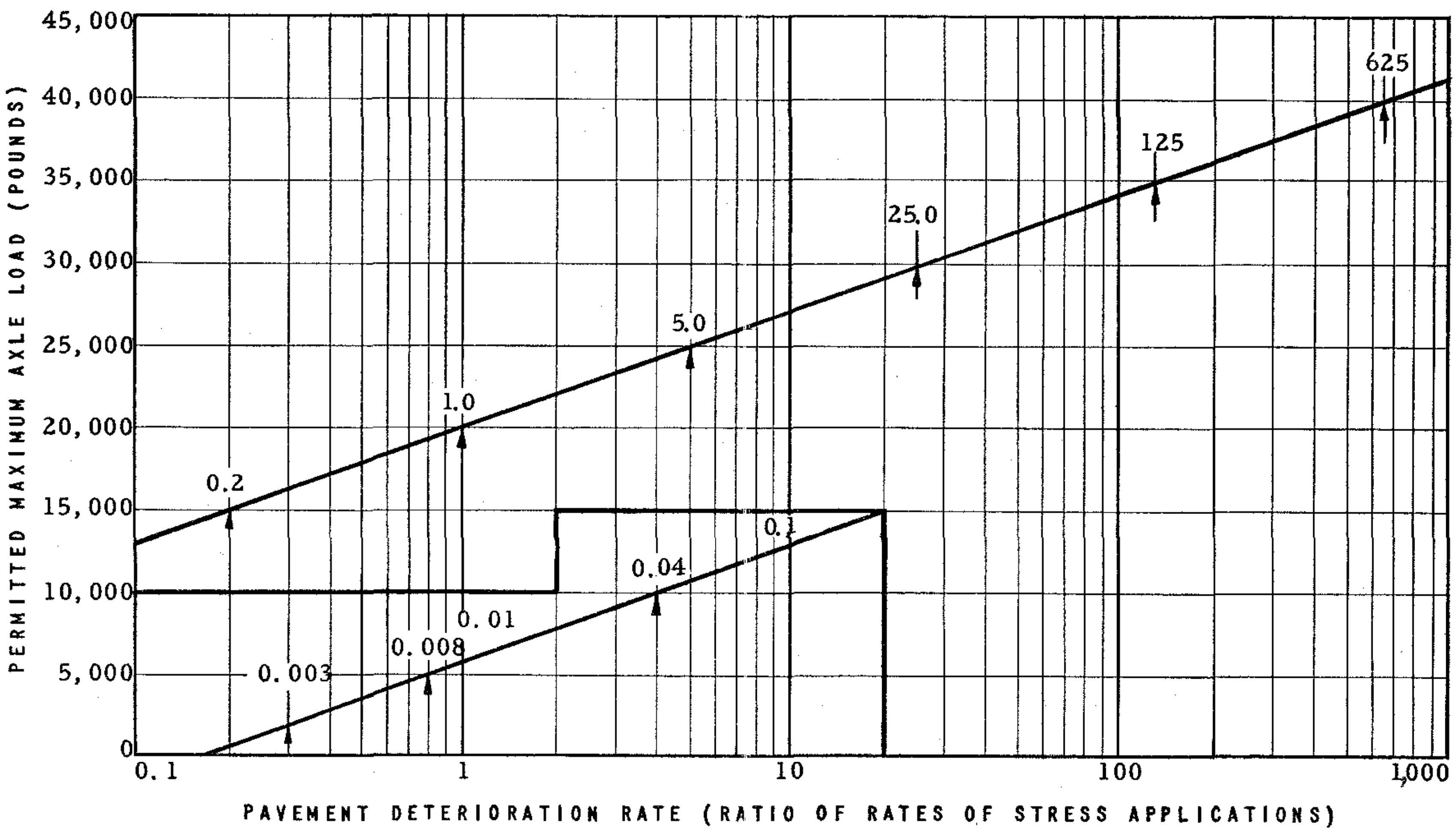
PAVEMENT DETERIORATION RELATIONSHIPS BETWEEN SPECIFIED WHEEL LOADS AND EQUIVALENT 5,000-POUND WHEEL LOADS*



* For Portland cement concrete pavements, 12 feet wide, with normal traffic distribution. Data averaged from curves for k = 100 and k = 200.

Chart VII

ESTIMATED PAVEMENT DETERIORATION RATES AT VARIOUS PERMITTED MAXIMUM AXLE LOADS*



* Based on 20,000-pound permitted maximum axle load.

tailed field studies, and they show average life corresponding to average subgrade support.

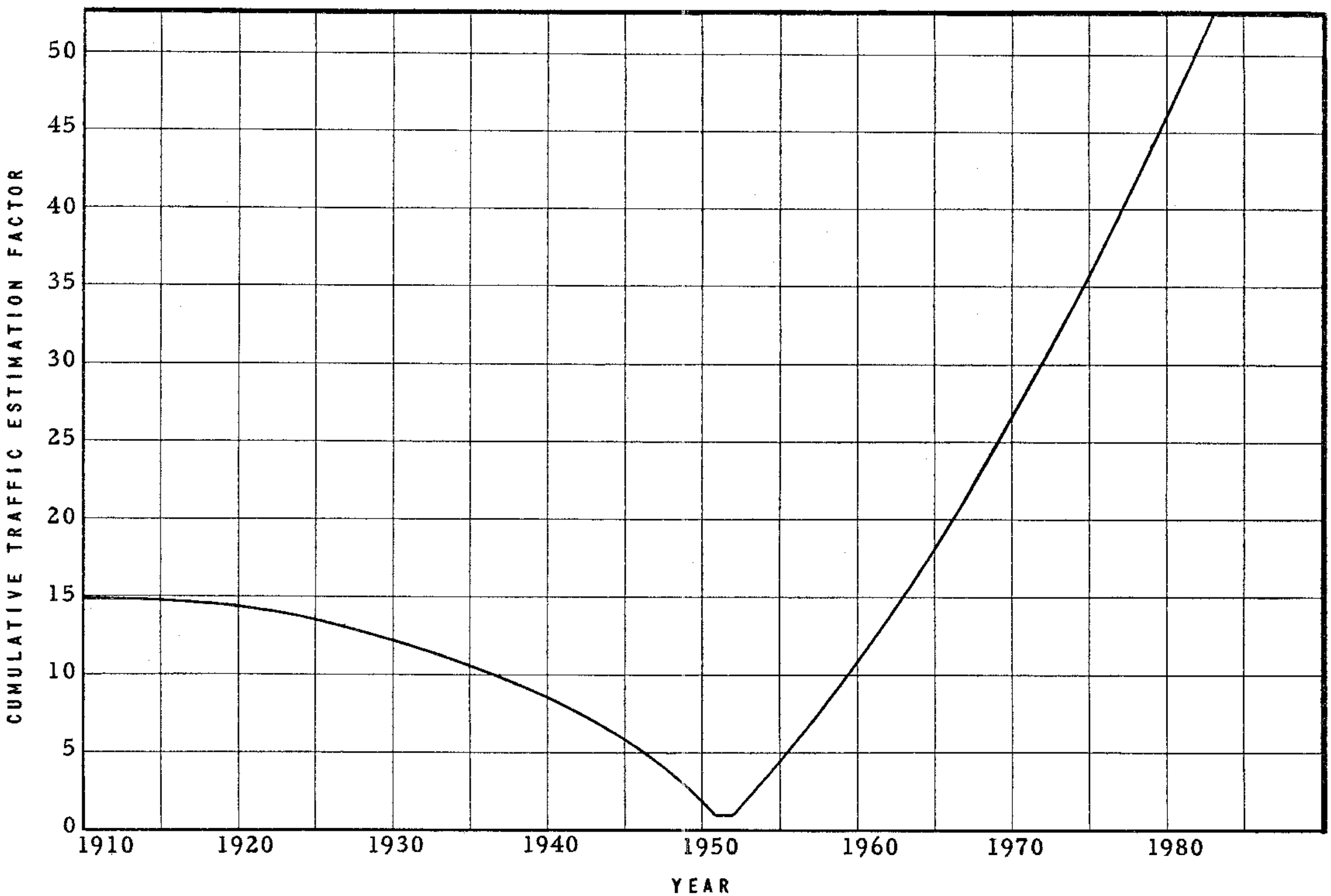
Remaining life, in years, was estimated for 128 sample highway sections. Tables 4 to 7, inclusive, summarize the changes in life resulting from modifications of existing axle loads (for the same total numbers of axles) which were determined by use of relationships shown in Chart VII. These data could be expanded to cover the highway system as a whole.

As a result of this study of pavement life, the following conclusions were reached:

1. Rational design of a highway for a specific life requires detailed knowledge of:
   a. Subgrade soil support
   b. Numbers and weights of axles
2. When these are known for a number of highways and the thickness and useful life have been determined, design methods relating subgrade characteristics, thickness and material, and life can be developed. Detailed records of highway performance are required for satisfactory results.
3. Field tests indicate that pavements may be designed and constructed for a specific life and axle loading for any type of subgrade.
4. The deteriorating effect of any wheel load may be expressed with reasonable accuracy in terms of a number of applications of a standard wheel load.

Chart VIII

CUMULATIVE TRAFFIC ESTIMATION FACTORS
FOR 1951 AVERAGE DAILY TRAFFIC*



* Derived from gasoline consumption and motor vehicle registration data.

Chart IX

COMPARISON OF ESTIMATED REMAINING LIFE, IN YEARS, FROM DESIGN METHOD COMPUTATION AND FIELD DATA

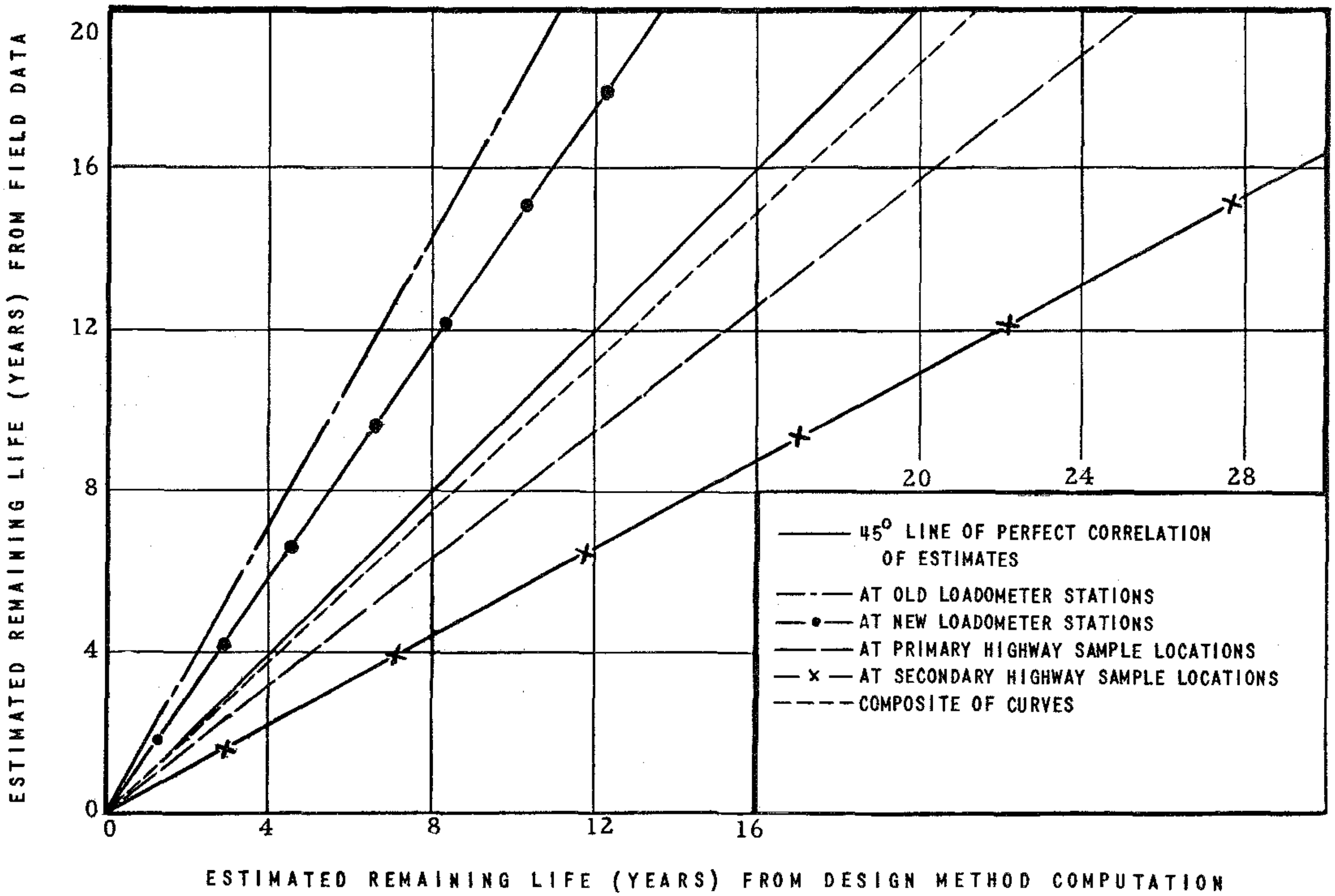


Table 4

ESTIMATED PAVEMENT LIFE, IN YEARS, AT DIFFERENT MAXIMUM AXLE-LOAD LIMITS: NEW LOADOMETER STATIONS

	Traffic Data		*Redesign Feasible*			*Changes Required (1, Light to 5, Heavy)*			*Estimated Life Remaining (Years) at Different Maximum Axle-Load Limits*								
Loadometer Station Number	*Average Daily Traffic (1951)*	*Percent Commercial*	*Yes*	*No*	*No Change Required*	*Alignment*	*Earthwork*	*Grade Reduction*	*5,000 Pounds*	*10,000 Pounds*	*15,000 Pounds*	*18,000 Pounds*	*20,000 Pounds*	*25,000 Pounds*	*30,000 Pounds*	*35,000 Pounds*	*40,000 Pounds*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
12	4,850	22%	..	..	x	..	..	..	608	126	30	14	8	2.0	0.4	0.08	0.02
53	3,800	19	x	..	..	1	1	1	218	49	13	5	3	0.7	0.1	0.03	0.005
59	2,750	17	..	x	..	..	..	..	292	63	17	7	4	0.9	0.2	0.04	0.007
88	3,200	18	x	..	..	3	3	3	292	63	17	7	4	0.9	0.2	0.04	0.007
100	7,750	27	x	..	..	1	1	1	260	56	15	6	3	0.8	0.2	0.03	0.006
104	7,150	18	x	..	..	2	3	3	965	196	41	20	12	2.8	0.6	0.12	0.02
110	7,700	28	..	..	x	..	..	..	450	83	24	11	6	1.3	0.3	0.06	0.01
123*	10,700	33	..	..	x	..	..	..	...	..	..	..	..	...	..	...	...
132	7,300	21	..	..	x	..	..	..	69	18	4	2	1	0.2	0.04	0.01	0.002
136	15,900	23	..	..	x	..	..	..	530	109	27	12	7	1.5	0.3	0.07	0.013
169*	4,950	27	..	x	..	..	..	..	...	..	..	..	..	...	..	...	...
182*	15,400	26	..	..	x	..	..	..	...	..	..	..	..	...	..	...	...
192	8,500	17	x	..	..	1	1	2	406	86	22	10	6	1.2	0.3	0.05	0.010
202*	5,500	28	x	..	..	1	2	2	...	..	..	..	..	...	..	...	...
461	8,000	20	x	..	..	4	4	3	378	79	20	9	5	1.1	0.2	0.05	0.009
503	5,200	16	..	..	x	..	..	..	978	198	45	20	12	2.8	0.6	0.12	0.02

*Incomplete data.

Table 5

Estimated Pavement Life, in Years, at Different Maximum Axle-Load Limits: Old Loadometer Stations

	Traffic Data		*Redesign Feasible*			*Changes Required (1, Light to 5, Heavy)*			*Estimated Life Remaining (Years) at Different Maximum Axle-Load Limits*								
Station Number	*Average Daily Traffic (1951)*	*Percent Commercial*	*Yes*	*No*	*No Change Required*	*Alignment*	*Earthwork*	*Grade Reduction*	*5,000 Pounds*	*10,000 Pounds*	*15,000 Pounds*	*18,000 Pounds*	*20,000 Pounds*	*25,000 Pounds*	*30,000 Pounds*	*35,000 Pounds*	*40,000 Pounds*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
1*	12,400	31%	..	x	..	..	..	..	...	...	..	..	..	...	...	...	...
2	2,200	29	x	..	..	1	1	1	187	42	11	5	2	0.6	0.12	0.02	0.005
4	4,100	23	x	..	..	3	3	2	142	33	9	4	2	0.4	0.09	0.02	0.004
32	3,100	28	x	..	..	1	1	1	375	79	20	9	5	1.1	0.23	0.05	0.009
94*	7,600	23	x	..	..	2	2	2	...	...	..	..	..	...	...	...	...
159*	1,650	25	x	..	..	2	3	3	...	...	..	..	..	...	...	...	...
171*	4,350	19	x	..	..	1	3	3	796	161	37	17	10	2.2	0.50	0.10	0.020
264	4,800	17	..	..	x	..	..	..	705	153	33	15	9	2.0	0.43	0.09	0.018
291	2,175	20	..	..	x	..	..	..	375	79	20	9	5	1.1	0.23	0.05	0.009
300*	2,900	21	..	x	..	..	..	..	...	...	..	..	..	...	...	...	...
301	9,900	18	x	..	..	2	3	2	380	79	20	9	5	1.1	0.23	0.05	0.009
302	13,200	18	x	..	..	1	1	1	435	93	24	11	6	1.3	0.28	0.06	0.011
437*	3,500	20	..	..	x	..	..	..	...	...	..	..	..	...	...	...	...
502	11,000	35	..	x	..	..	..	..	2,680	410	87	38	22	5.5	1.22	0.26	0.051

*Incomplete data.

Table 6

ESTIMATED PAVEMENT LIFE, IN YEARS, AT DIFFERENT MAXIMUM AXLE-LOAD LIMITS: SELECTED PRIMARY ROUTE STATIONS

	Traffic Data		Redesign Feasible			Changes Required (1, Light to 5, Heavy)			Estimated Life Remaining (Years) at Different Maximum Axle-Load Limits								
Station Number	*Average Daily Traffic (1951)*	*Percent Commercial*	*Yes*	*No*	*No Change Required*	*Alignment*	*Earthwork*	*Grade Reduction*	*5,000 Pounds*	*10,000 Pounds*	*15,000 Pounds*	*18,000 Pounds*	*20,000 Pounds*	*25,000 Pounds*	*30,000 Pounds*	*35,000 Pounds*	*40,000 Pounds*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
1	4,888	21%	x	..	..	2	2	2	219	49	13	6	3	0.7	0.14	0.03	0.005
2	4,206	15	x	..	..	1	1	1	795	160	37	17	10	2.2	0.50	0.10	0.020
5*	7,289	21	..	..	x	..	..	..	...	...	..	..	..	...	...	...	....
6	5,780	22	x	..	..	5	4	3	69	18	4	2	1	0.2	0.04	0.01	0.002
16	2,206	14	x	..	..	4	4	4	375	79	20	9	5	1.1	0.23	0.05	0.009
17	335	14	x	..	..	3	3	3	181	41	11	5	2	0.6	0.11	0.02	0.004
19	5,505	24	x	..	..	3	3	2	225	49	13	6	3	0.7	0.13	0.03	0.005
23	918	15	x	..	..	4	3	2	219	49	13	6	3	0.7	0.14	0.03	0.005
25*	2,724	16	x	..	..	5	5	5	...	...	..	..	..	...	...	...	...
28*	5,417	18	x	..	..	3	2	2	...	...	..	..	..	...	...	...	...
31	793	16	x	..	..	2	3	2	615	127	30	14	8	1.8	0.39	0.08	0.015
32*	4,503	23	x	..	..	5	5	5	...	...	..	..	..	...	...	...	...
33	3,435	16	..	..	x	..	..	..	1,065	217	48	22	13	3.0	0.67	0.13	0.027
39	1,780	18	x	..	..	1	1	1	290	57	16	7	4	0.9	0.18	0.04	0.007
41	2,374	13	..	x	..	..	..	..	615	127	29	14	8	1.8	0.39	0.08	0.015
44	2,477	17	x	..	..	1	1	1	970	197	45	20	12	2.8	0.62	0.12	0.025
47*	1,001	21	x	..	..	2	2	2	...	...	..	..	..	...	...	...	...
50*	661	18	x	..	..	2	3	2	...	...	..	..	..	...	...	...	...
51*	2,312	15	x	..	..	3	3	2	...	...	..	..	..	...	...	...	...
53*	5,573	21	x	..	..	5	3	3	...	...	..	..	..	...	...	...	...
54	671	21	..	x	..	..	..	..	142	34	9	4	2	0.4	0.09	0.02	0.004
55	1,789	15	x	..	..	1	2	1	540	109	27	12	7	1.5	0.33	0.07	0.013
56	1,211	20	x	..	..	1	2	1	705	142	34	16	9	2.0	0.48	0.1	0.019
58*	8,258	24	x	..	..	..	..	..	69	18	4	2	1	0.2	0.04	0.01	0.002
63	4,943	16	x	..	..	1	3	2	69	18	4	2	1	0.2	0.04	0.01	0.002

*Incomplete data.

Table 6 (Continued)

Station Number	Traffic Data: Average Daily Traffic (1951)	Traffic Data: Percent Commercial	Redesign Feasible: Yes	Redesign Feasible: No	No Change Required	Changes Required (1, Light to 5, Heavy): Align-Ment	Changes Required: Earth-work	Changes Required: Grade Reduc-tion	Estimated Life Remaining (Years) at Different Maximum Axle-Load Limits: 5,000 Pounds	10,000 Pounds	15,000 Pounds	18,000 Pounds	20,000 Pounds	25,000 Pounds	30,000 Pounds	35,000 Pounds	40,000 Pounds
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
65	6,606	15	x	..	..	1	2	2	142	34	9	4	2	0.4	0.09	0.02	0.004
67*	1,321	17	x	..	..	2	5	4	...	...	..	..	..	...	...	...	...
82	451	15	x	..	..	3	3	3	615	127	30	14	8	1.7	0.38	0.08	0.015
83	3,193	22	..	..	x	..	..	..	795	158	37	17	10	2.2	0.50	0.09	0.020
90*	1,101	20	x	..	..	4	3	3	...	...	..	..	..	...	...	...	...
91*	10,680	25	..	x	..	..	..	..	...	...	..	..	..	...	...	...	...
94	3,920	17	..	..	x	..	..	..	793	161	38	17	10	2.2	0.50	0.10	0.020
98	771	16	..	x	..	..	..	..	382	77	20	9	5	1.1	0.23	0.05	0.009
101	1,652	24	x	..	..	3	3	3	69	18	4	2	1	0.2	0.04	0.01	0.002
113	1,889	15	x	..	..	4	1	1	793	161	37	17	10	2.2	0.50	0.10	0.020
135*	5,000	22	x	..	..	3	2	1	...	...	..	..	..	...	...	...	...
139	617	18	x	..	..	3	2	3	535	109	27	12	7	1.5	0.33	0.07	0.013
140	771	18	x	..	..	3	2	3	455	93	24	11	6	1.3	0.28	0.06	0.011
141	3,826	22	x	..	..	2	3	1	70	18	5	2	1	0.2	0.04	0.01	0.002
143	165	13	..	x	..	..	..	..	390	79	20	9	5	1.1	0.23	0.05	0.009
145*	12,386	24	..	..	x	..	..	..	...	...	..	..	..	...	...	...	...
146	2,488	20	x	..	..	3	2	1	615	127	30	14	8	1.7	0.38	0.08	0.015
148	1,020	18	x	..	..	1	1	1	615	127	30	14	8	1.7	0.38	0.08	0.015
151*	2,532	13	x	..	..	4	3	3	...	...	..	..	..	...	...	...	...
156	782	20	..	x	..	..	..	..	970	198	45	20	12	2.8	0.61	0.12	0.024
158	2,028	22	x	..	..	1	1	2	1,030	218	49	22	13	3.0	0.67	0.14	0.027
164*	4,090	18	x	..	..	2	5	3	...	...	..	..	..	...	...	...	...
167	3,039	12	..	..	x	..	..	..	2,230	380	76	34	20	4.9	1.11	0.23	0.046
176	176	12	..	x	..	..	..	..	885	178	36	19	11	2.5	0.55	0.11	0.022
178*	5,505	18	x	..	..	1	2	2	...	...	..	..	..	...	...	...	...

*Incomplete data.

Table 7

Estimated Pavement Life, in Years, at Different Maximum Axle-Load Limits: Selected Secondary Route Stations

	Traffic Data		*Redesign Feasible*			*Changes Required (1, Light to 5, Heavy)*			*Estimated Life Remaining (Years) at Different Maximum Axle-Load Limits*								
Station Number	*Average Daily Traffic (1951)*	*Percent Commercial*	*Yes*	*No*	*No Change Required*	*Alignment*	*Earthwork*	*Grade Reduction*	*5,000 Pounds*	*10,000 Pounds*	*15,000 Pounds*	*18,000 Pounds*	*20,000 Pounds*	*25,000 Pounds*	*30,000 Pounds*	*35,000 Pounds*	*40,000 Pounds*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
1	539	15%	..	x	..	..	..	..	530	109	27	12	7	1.5	0.33	0.07	0.013
9*	4,503	16	x	..	..	1	5	1	...	...	..	..	..	...	...	...	...
23	622	20	..	x	..	..	..	..	...	...	..	..	..	...	...	...	...
29	475	16	x	..	..	3	3	2	290	63	17	7	4	0.9	0.18	0.04	0.007
30	1,042	16	x	..	..	4	4	3	...	...	..	..	..	...	...	...	...
45*	716	15	..	x	..	..	..	..	...	...	..	..	..	...	...	...	...
46	396	14	x	..	..	4	3	3	530	109	27	12	7	1.5	0.33	0.07	0.013
62	165	17	x	..	..	4	4	4	795	162	37	18	10	2.3	0.50	0.10	0.020
67†	83	14	x	..	..	5	5	5	...	...	..	..	..	...	...	...	...
80	361	13	..	x	..	..	..	..	795	162	37	18	10	2.3	0.50	0.10	0.020
84	881	17	..	..	x	..	..	..	615	127	30	14	8	1.8	0.39	0.08	0.015
85	842	17	..	..	x	..	..	..	615	127	30	14	8	1.7	0.38	0.08	0.015
88	186	14	x	..	..	5	2	1	795	162	37	17	10	2.2	0.50	0.10	0.020
90†	61	13	x	..	..	5	5	3	...	...	..	..	..	...	...	...	...
94	726	14	x	..	..	2	4	1	219	49	13	6	3	0.7	0.14	0.03	0.005
101	551	18	..	x	..	..	..	..	455	94	24	10	6	1.3	0.28	0.06	0.011
103	1,101	16	..	x	..	..	..	..	142	34	9	4	2	0.4	0.09	0.02	0.004
104	881	14	..	x	..	..	..	..	2,840	420	91	39	23	5.9	1.29	0.27	0.054
107*	1,032	17	x	..	..	2	1	1	...	...	..	..	..	...	...	...	...
123	275	16	..	x	..	..	..	..	615	127	30	14	8	1.7	0.39	0.08	0.015
128	457	14	x	..	..	4	3	3	142	34	9	4	2	0.4	0.09	0.02	0.004
140‡	66	14	..	x	..	..	..	..	...	...	..	..	..	...	...	...	...
141	129	16	x	..	..	2	2	1	795	162	37	17	10	2.2	0.49	0.10	0.020
146†	55	13	x	..	..	5	5	5	...	...	..	..	..	...	...	...	...

*Incomplete data.
†Gravel surface.
‡Shale surface.

Table 7 (Continued)

Station Number	Traffic Data: Average Daily Traffic (1951)	Traffic Data: Percent Commercial	Redesign Feasible: Yes	Redesign Feasible: No	No Change Required	Changes Required (1, Light to 5, Heavy): Alignment	Changes Required: Earthwork	Changes Required: Grade Reduction	Estimated Life Remaining (Years) at Different Maximum Axle-Load Limits: 5,000 Pounds	10,000 Pounds	15,000 Pounds	18,000 Pounds	20,000 Pounds	25,000 Pounds	30,000 Pounds	35,000 Pounds	40,000 Pounds
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
162*	440	16	x	..	..	5	4	3	...	...	..	..	..	...	...	...	...
163	495	16	x	..	..	5	3	3	610	126	30	14	8	1.7	0.38	0.08	0.015
179	1,762	11	..	x	..	..	..	..	610	126	30	14	8	1.7	0.38	0.08	0.015
214	407	18	x	..	..	3	3	3	455	94	24	11	6	1.3	0.28	0.06	0.011
215	495	12	x	..	..	3	3	3	610	126	30	14	8	1.7	0.38	0.08	0.015
220	2,500	28	x	..	..	1	1	1	295	63	17	7	4	0.9	0.18	0.04	0.007
229	227	15	x	..	..	1	1	1	1,660	260	56	25	15	3.5	0.80	0.16	0.036
232	2,918	17	x	..	..	2	1	1	219	49	13	6	3	0.7	0.14	0.03	0.005
233	495	16	x	..	..	3	3	4	795	165	38	17	10	2.2	0.50	0.10	0.020
234	1,255	18	x	..	..	3	3	2	530	109	27	12	7	1.5	0.33	0.06	0.013
243	220	15	..	x	..	..	..	..	2,080	330	68	30	18	4.4	1.00	0.20	0.040
255	55	11	x	..	..	1	3	3	610	126	30	14	8	1.8	0.38	0.08	0.015
258†	387	16	..	x	..	..	..	..	...	...	..	..	..	...	...	...	...
265	360	17	x	..	..	3	1	1	288	63	17	7	4	0.9	0.18	0.03	0.007
270	853	15	x	..	..	1	2	2	610	126	30	14	8	1.8	0.38	0.08	0.015
271†	209	12	..	x	..	..	..	..	...	...	..	..	..	...	...	...	...
273	793	13	..	..	x	..	..	..	610	126	30	14	8	1.8	0.38	0.08	0.015
281	220	15	x	..	..	3	3	3	380	79	20	9	5	1.1	0.23	0.05	0.009
288*	41	10	..	x	..	..	..	..	...	...	..	..	..	...	...	...	...
291	176	19	..	x	..	..	..	..	610	126	30	14	8	1.7	0.38	0.08	0.015
293*	248	16	..	x	..	..	..	..	...	...	..	..	..	...	...	...	...
297*	606	15	x	..	..	3	3	3	...	...	..	..	..	...	...	...	...
301*	826	15	x	..	..	5	5	4	...	...	..	..	..	...	...	...	...
307	385	14	x	..	..	2	2	2	455	94	24	11	6	1.3	0.28	0.06	0.011

*Incomplete data.
†Gravel surface.

Section II
HIGHWAY DESIGN

Methods of highway design differ among the states and, in varying degree, depend on highway theory and highway experience. A few states utilize designs based wholly on theory. In these cases, a set of equations, based on the theory of elasticity, is the basis of the design. Soil evaluation factors obtained in the field or laboratory are used in these equations. Design procedures based wholly on theory have been satisfactory in certain types of engineering construction, but available theories for highways are inadequate for actual construction because subgrade soils are not entirely uniform or homogeneous and methods for evaluating subgrade strength are imperfect. In addition, there is considerable doubt that subgrade soil behaves elastically when high strains are applied, as assumed in most equations.

A common method of highway design is based predominantly on experience. This method may be satisfactory if a new pavement is to be built in an area where subgrade conditions are well known and reasonably uniform and where both traffic volumes and vehicle weights are not appreciably different from others previously observed. Such a method of design can be used when quantifications of all factors simultaneously duplicate past conditions. However, subgrade support is generally not constant under the entire length of a highway, and adequate design based on experience must include removal or improvement of questionable subgrades. This design procedure contains elements of uncertainty. When subgrades under any new road turn out to be poorer than expected, the road may deteriorate far more rapidly than anticipated. For this reason, care should be used to provide adequate subgrade support for all highways.

Pavements may be built to high standards in all other respects and yet develop serious cracking, pumping, and other types of deterioration within a few years if the subgrade design is faulty. Failures require subsealing and other costly maintenance treatments. A six-inch layer of special subgrade is often used to provide drainage and strength. This will improve subgrade support to some degree, but there is no assurance that this treatment will be adequate to supply the necessary additional support under all new pavements.

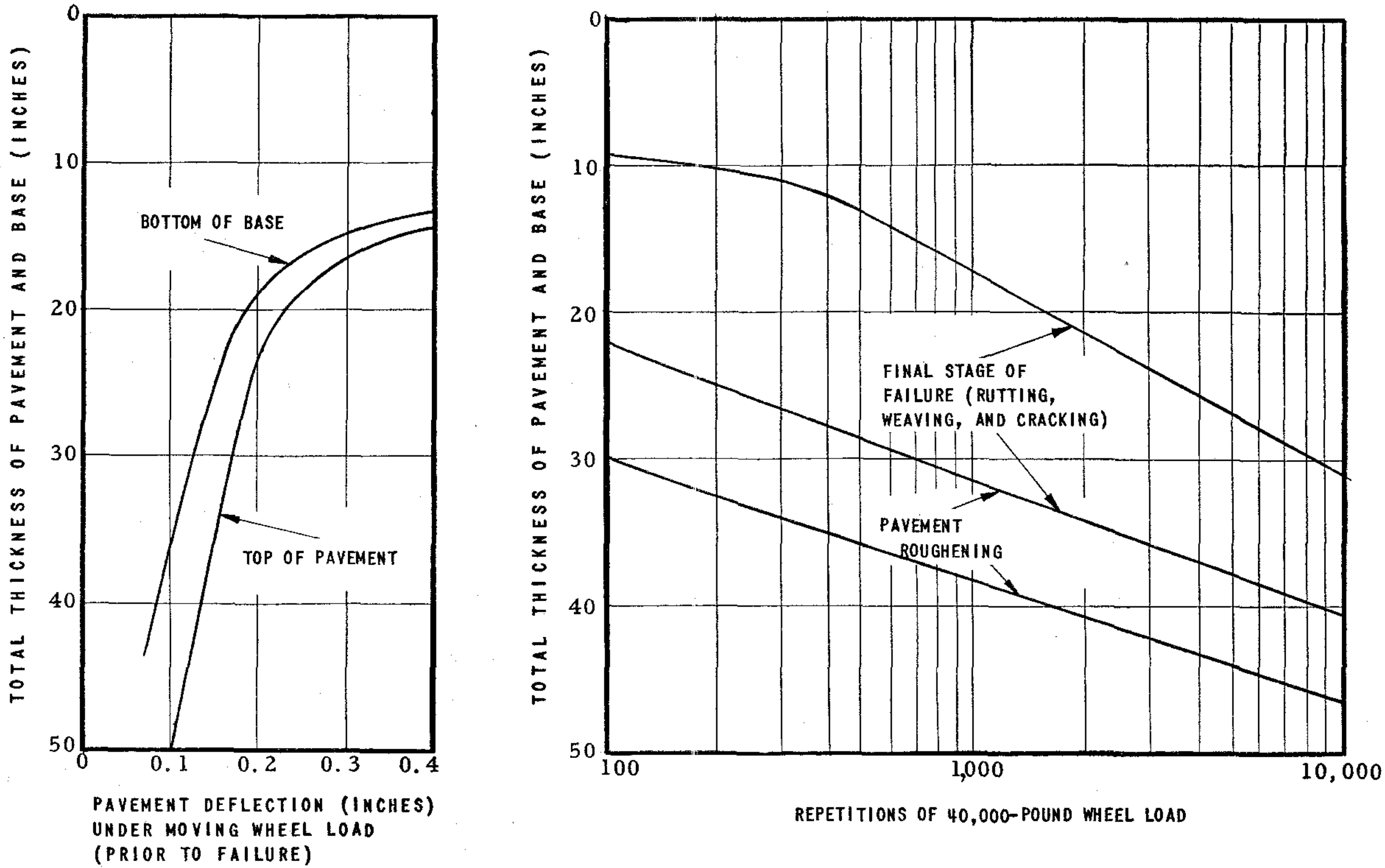
From the foregoing, it appears that if highway design theory, alone, and experience, alone, are inadequate, the alternative is to base design practices on a combination of the two. Consequently, in most design procedures, a semiempirical approach combining past practical experience and theory is used. In this method, the problem is separated into its components: stress repetitions, pavement thickness, and subgrade support. Highways near the end of their useful lives are studied to determine the number of repetitions which were carried (given pavement design and subgrade support). These data establish certain distinct relationships which permit designs for any number of stress repetitions on any pavement or base. This design approach, combining experience with theory, appears to have great promise.

A. Flexible Pavements

The California Bearing Ratio method for establishing the quality of natural subgrade, as well as of base and subbase, was devised to overcome certain objections to the field plate loading tests (B-22). Performance data for California highways with light and medium traffic also were compiled over a period of twelve years, and from

Chart X

PAVEMENT DEFLECTION AND RATE OF PAVEMENT FAILURE UNDER 40,000-POUND WHEEL LOADS*



* Total thickness consists of 3-inch depth of asphaltic concrete pavement plus variable thicknesses of crushed gravel base.

Source: *Proceedings*, Highway Research Board, Vol. 22 (1942), p. 122, ff.

these data the California type A and B design curves were derived (B-21). These curves were supplemented by detailed field studies of test pavements (to complete deterioration), from which a set of curves relating pavement thickness to subgrade support for a satisfactory life was developed. No specific number of equivalent wheel-load repetitions is cited. Apparently most of the higher loading curves were designed for a life of about 20,000 repetitions of the maximum loading, but in this high range it was considered inadvisable to extrapolate the 1:2:4 E.W.L. series to obtain life predictions. One of the relationships developed from the California study is presented as Chart X. It may be noted that, at 500 repetitions of the 40,000-pound load, a 12-inch total depth of flexible pavement and base would have been completely destroyed. The addition of 16 inches of special subgrade improved the subgrade support so that the pavement, although rutted and cracked, was still passable. However, the addition of 8 inches more of select subgrade improved support so that 500 passes of the 40,000-pound wheel load only roughened the pavement. A total design thickness of 40 inches resulted in a pavement life 16 times as great as that of a highway with a total depth of 28 inches and 200 times that of one with a 12-inch depth.

If subgrades are designed for high minimum supporting capacity, flexible pavements can be built to last for any desired life under any magnitude of axle loading. Procedures for relating pavement deterioration to thickness of pavement and quality of subgrade, and taking into account numbers and weights of axles, were studied. Test samples of highways in Pennsylvania which had reached the end of their useful life were selected, and the number of 5,000-pound equivalent wheel loadings for each was computed. An approach to the development of original design curves using these data (together with pavement thickness and subgrade C.B.R. information) proved impractical, since insufficient data were available. However, adequate data were available from a detailed study made in Kentucky in 1947 and 1948 (B-2). The methods employed for evaluating subgrade support included field and laboratory C.B.R. tests, 12- and 30-inch diameter plate tests, and the North Dakota Cone Test. The method used for evaluating the 5,000-pound equivalent wheel-load repetitions was based on the geometric series previously described. A set of curves relating equivalent wheel-load repetitions and pavement thickness was presented for each of the methods of measuring subgrade support.

Though there were variations, the differences in accuracy among these five sets of curves were not appreciable. For correlation with Pennsylvania highway life studies, an average of these curves, excluding the North Dakota Cone Curve, was used. (The North Dakota Cone curve was not included since it is difficult to correlate with C.B.R. and subgrade modulus values.) Examples of these curves are shown in Charts XI and XII. After study and comparison with data from airfield pavement tests by U. S. Army Engineers, design thicknesses were modified slightly for weak subgrades and for very good subgrades. The design curve selected for further check under Pennsylvania conditions is shown in Chart XIII.[1]

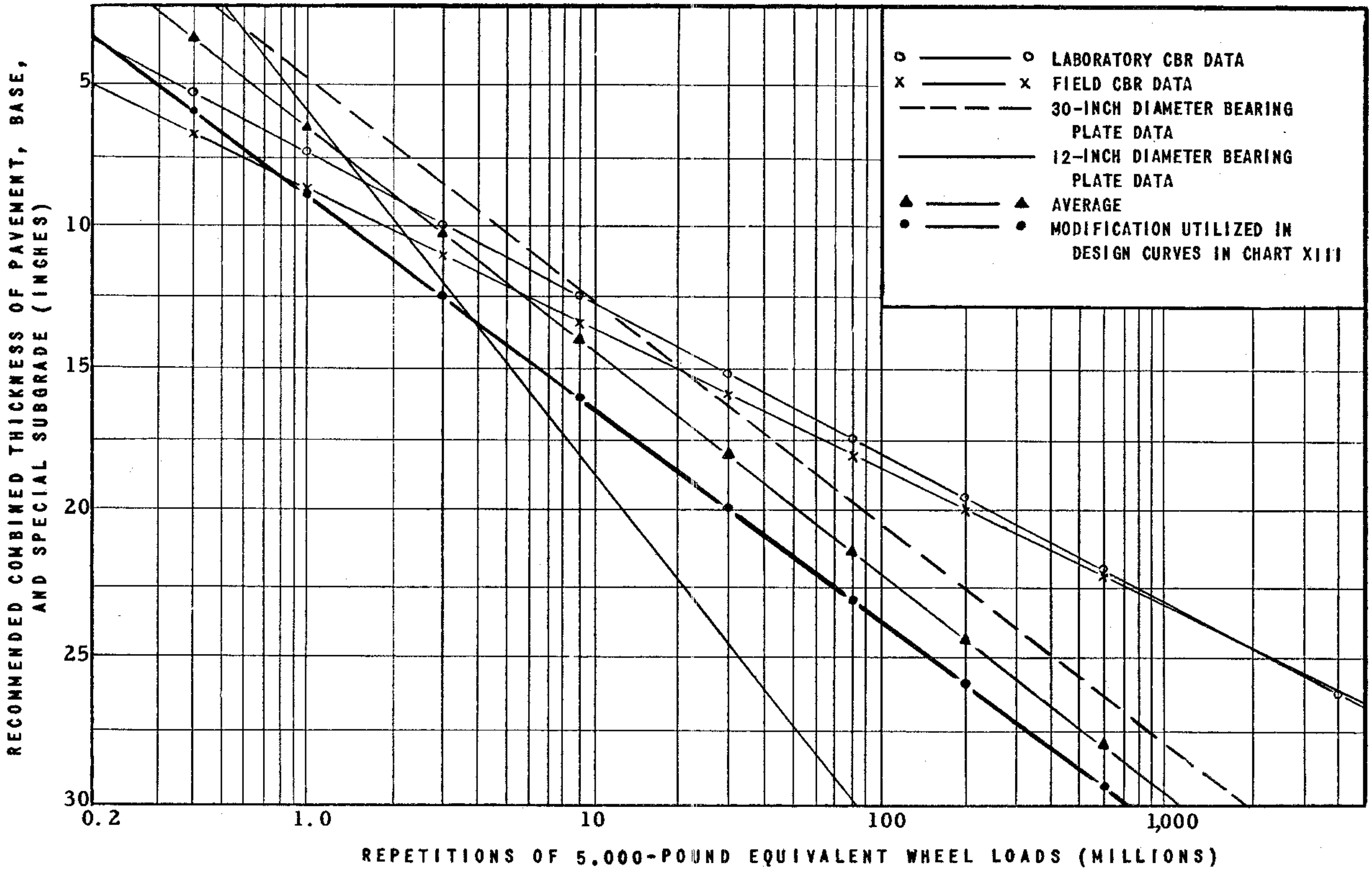
The curves shown in Chart XIII and the design data shown in Tables 8 and 9 obtained therefrom, relate to design of new flexible highways and overlays on existing highways and were prepared solely for estimating purposes. They are not intended for other use.

Compensation for natural subgrade support deficiencies is implicit in the procedures and curves for pavement design. Certain additional data illustrating advantages of added subgrade thickness are presented in Charts XIV and XV. These data represent preliminary results of research by the Civil Aeronautics Authority on relationships of

[1] For geometric design standards, see Appendix A.

Chart XI

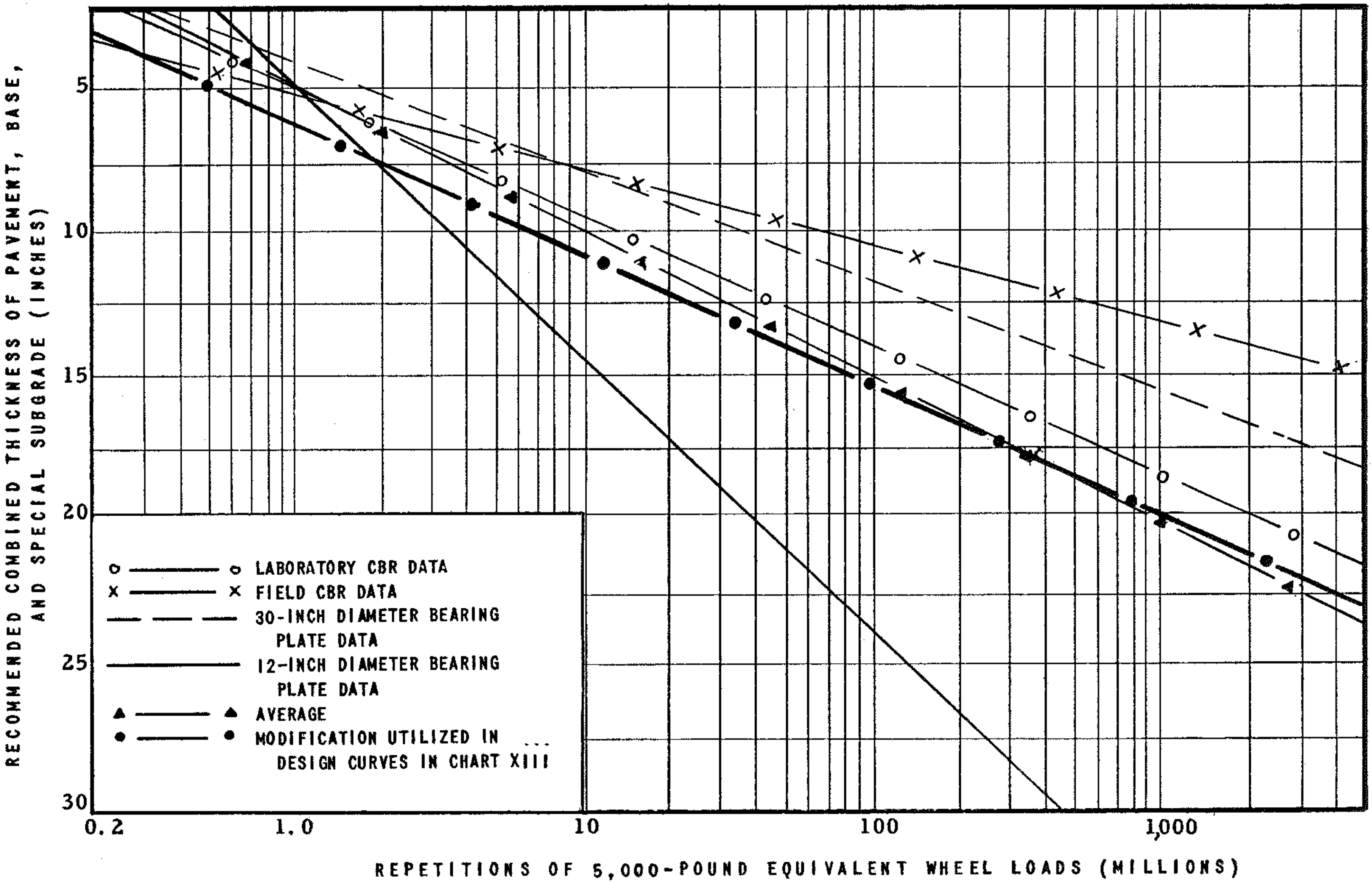
RELATIONSHIP FOR C.B.R. = 7.5, BETWEEN COMBINED THICKNESS OF PAVEMENT, BASE, AND SPECIAL SUBGRADE AND REPETITIONS OF 5,000-POUND EQUIVALENT WHEEL LOADS



Source: *Proceedings*, Highway Research Board, Vol. 28 (1948), pp. 69, 72.

Chart XII

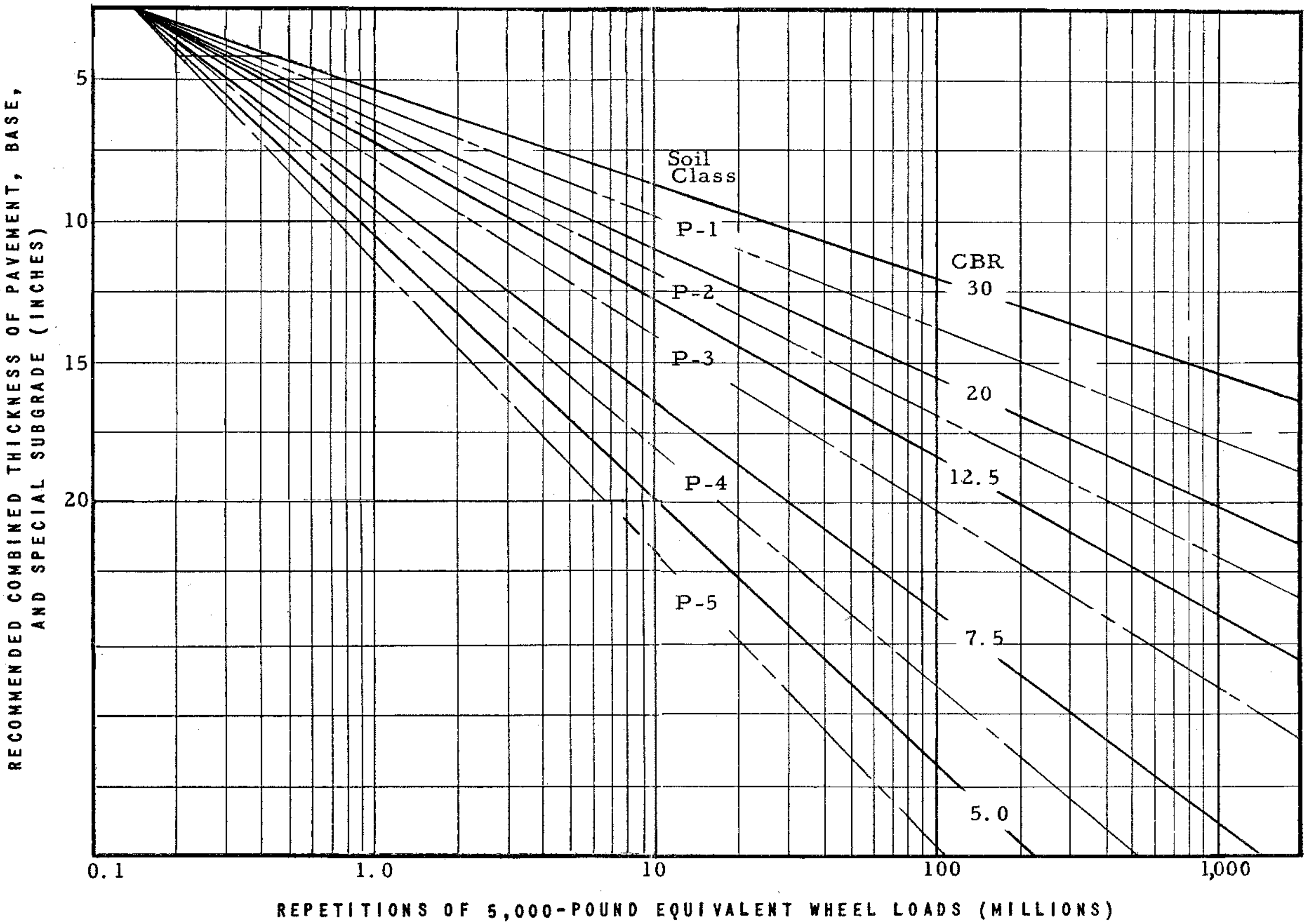
RELATIONSHIP FOR C.B.R. = 20, BETWEEN COMBINED THICKNESS OF PAVEMENT, BASE, AND SPECIAL SUBGRADE AND REPETITIONS OF 5,000-POUND EQUIVALENT WHEEL LOADS



Source: *Proceedings*, Highway Research Board, Vol. 28 (1948), pp. 69, 72.

Chart XIII

FLEXIBLE PAVEMENT DESIGN CURVES RELATING RECOMMENDED COMBINED THICKNESS OF PAVEMENT, BASE, AND SPECIAL SUBGRADE AND REPETITIONS OF 5,000-POUND EQUIVALENT WHEEL LOADS, FOR SUBGRADE SOILS OF VARIOUS CLASSIFICATIONS AND BEARING CAPACITIES*



* Thickness of asphaltic concrete, crushed aggregate base, and special subgrade recommended for each combined thickness are shown in Tables 8 and 9.

Table 8

COMBINED THICKNESS OF BITUMINOUS CONCRETE, CRUSHED AGGREGATE BASE AND SELECT SUBGRADE RECOMMENDED FOR 25-YEAR PAVEMENT LIFE AT SPECIFIED MAXIMUM AXLE-LOAD LIMITS

		Combined Thickness (Inches) Recommended for 25-Year Life						
		Pennsylvania Department of Highways Traffic Classification						
Maximum Axle-load Limit (Pounds)	*Natural Subgrade Soil Classification*	*1*	*2*	*3*	*4*	*5*	*6*	*7*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
2,000	1	6.0	4.5	4.0	4.0	4.0	4.0	4.0
	2	6.5	5.0	4.5	4.5	4.5	4.5	4.5
	3	7.5	5.5	5.0	5.0	5.0	5.0	5.0
	4	9.5	6.5	6.0	6.0	6.0	6.0	6.0
	5	11.0	7.5	7.0	7.0	7.0	7.0	7.0
5,000	1	7.5	6.0	4.0	4.0	4.0	4.0	4.0
	2	8.5	7.0	4.5	4.5	4.5	4.5	4.5
	3	10.0	8.0	5.0	5.0	5.0	5.0	5.0
	4	12.5	9.5	6.0	6.0	6.0	6.0	6.0
	5	15.0	11.5	7.0	7.0	7.0	7.0	7.0
10,000	1	10.0	8.5	7.0	4.0	4.0	4.0	4.0
	2	12.0	10.5	8.0	4.5	4.5	4.5	4.5
	3	14.5	12.5	9.5	5.0	5.0	5.0	5.0
	4	18.5	15.5	11.5	6.0	6.0	6.0	6.0
	5	25.0	18.5	13.5	7.0	7.0	7.0	7.0
15,000	1	13.0	11.0	9.5	7.0	5.0	4.0	4.0
	2	16.0	13.5	11.5	8.0	6.0	4.5	4.5
	3	19.0	16.0	13.5	9.5	7.0	5.0	5.0
	4	24.0	21.5	17.0	11.5	8.5	6.0	6.0
	5	29.5	26.0	21.0	13.5	10.0	7.0	7.0
20,000	1	16.0	14.0	12.0	9.0	8.0	6.0	4.0
	2	18.0	18.0	15.0	11.0	9.5	7.0	4.5
	3	23.0	21.0	18.0	13.0	11.0	8.0	5.0
	4	30.0	28.0	23.0	16.0	14.0	9.5	6.0
	5	36.0	33.0	27.0	20.0	17.0	11.5	7.0
25,000	1	19.0	17.0	15.0	12.0	11.0	8.5	7.0
	2	23.0	21.0	19.0	14.5	13.0	10.5	8.0
	3	27.0	25.0	23.0	17.0	15.0	12.5	9.5
	4	36.0	33.0	29.0	22.0	20.0	15.5	11.5
	5	43.0	40.0	36.0	27.0	24.0	18.5	13.5
30,000	1	21.0	20.0	18.0	14.5	13.5	11.5	9.5
	2	26.0	24.5	22.0	18.0	16.5	13.5	11.5
	3	32.0	29.5	27.0	21.5	19.5	16.0	13.5
	4	44.0	38.0	35.0	28.0	25.5	21.5	17.0
	5	56.0	47.0	43.0	34.0	31.0	26.0	21.0
35,000	1	23.5	22.0	20.5	17.0	16.0	14.0	12.0
	2	29.5	28.0	25.5	21.0	20.0	17.5	15.0
	3	36.0	34.0	31.0	26.0	24.0	21.0	18.0
	4	48.0	44.0	40.0	33.0	31.0	27.5	23.0
	5	58.0	55.0	50.0	41.0	38.0	33.0	28.0
40,000	1	26.0	25.0	23.0	20.0	19.0	17.0	15.0
	2	33.0	31.0	29.0	25.0	23.5	21.0	18.5
	3	40.0	38.0	35.0	30.0	28.0	25.0	22.0
	4	53.0	50.0	46.0	36.0	33.0	33.0	29.0
	5	65.0	61.0	57.0	48.0	46.0	40.0	35.0

NOTES

(a) If total thickness is 8″ or less, a 2″ hot-mix surface is recommended; 8″ to 12″, a 2½″; 12″ to 18″, a 3″; over 18″, a 3½″. However, if total thickness is less than 5″, a lower quality penetration or road mix is recommended.

(b) A 6″ mechanically stabilized, well-graded, crushed aggregate base course under all pavement 8″ or more in thickness is recommended; for base courses of lesser thickness, a 1½″ maximum size, well-graded, mechanically stabilized base.

(c) To increase supporting capacity of weak natural subgrades, lower quality materials can be utilized including quarry waste, pit run gravel, slag, certain gravelly clays, etc. If C.B.R. or equivalent test rating of material exceeds C.B.R. rating required at that depth below pavement surface, local material is acceptable. Part of the required material for deeper select subgrades may be local soil compacted to higher density.

(d) If existing bituminous concrete is to be strengthened, all existing depths of pavement, base, or special subgrade are considered, and the overlay consists of the added thickness necessary.

Table 9

THICKNESS OF BITUMINOUS CONCRETE AND CRUSHED AGGREGATE OVERLAYS OVER OLD CONCRETE PAVEMENT RECOMMENDED FOR 25-YEAR LIFE AT SELECTED MAXIMUM AXLE-LOAD LIMITS

		Thickness (Inches) of Bituminous Concrete and Crushed Aggregate Overlays Recommended for 25-Year Life						
Maximum Axle Load Limit (Pounds)	*Natural Subgrade Soil Classification*	*Pennsylvania Department of Highways Traffic Classification* 1	2	3	4	5	6	7
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
2,000	1	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	2	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	3	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	4	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	5	2.0	2.0	2.0	2.0	2.0	2.0	2.0
5,000	1	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	2	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	3	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	4	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	5	3.0	2.0	2.0	2.0	2.0	2.0	2.0
10,000	1	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	2	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	3	2.5	2.0	2.0	2.0	2.0	2.0	2.0
	4	6.5	3.5	2.0	2.0	2.0	2.0	2.0
	5	13.0	6.5	2.0	2.0	2.0	2.0	2.0
15,000	1	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	2	4.0	2.0	2.0	2.0	2.0	2.0	2.0
	3	7.0	4.0	2.0	2.0	2.0	2.0	2.0
	4	12.0	9.5	5.0	2.0	2.0	2.0	2.0
	5	17.5	14.0	9.0	2.0	2.0	2.0	2.0
20,000	1	4.0	2.0	2.0	2.0	2.0	2.0	2.0
	2	6.0	6.0	3.0	2.0	2.0	2.0	2.0
	3	11.0	9.0	6.0	2.0	2.0	2.0	2.0
	4	18.0	16.0	11.0	4.0	2.0	2.0	2.0
	5	24.0	21.0	15.0	8.0	5.0	2.0	2.0
25,000	1	7.0	5.0	3.0	2.0	2.0	2.0	2.0
	2	11.0	9.0	7.0	2.5	2.0	2.0	2.0
	3	15.0	13.0	11.0	5.0	3.0	2.0	2.0
	4	24.0	21.0	17.0	10.0	8.0	3.5	2.0
	5	31.0	28.0	24.0	15.0	12.0	6.0	2.0
30,000	1	9.0	8.0	6.0	2.5	2.0	2.0	2.0
	2	14.0	12.5	10.0	6.0	4.5	2.0	2.0
	3	20.0	17.5	15.0	9.5	7.5	4.0	2.0
	4	32.0	26.0	23.0	16.0	13.5	9.5	5.0
	5	44.0	35.0	31.0	22.0	19.0	14.0	9.0
35,000	1	11.5	10.0	8.5	5.0	4.0	2.0	2.0
	2	17.5	16.0	13.5	9.0	8.0	5.5	3.0
	3	24.0	22.0	19.0	14.0	12.0	9.0	6.0
	4	36.0	32.0	28.0	21.0	19.0	15.5	11.0
	5	46.0	43.0	38.0	29.0	26.0	21.0	16.0
40,000	1	14.0	13.0	11.0	8.0	7.0	5.0	3.0
	2	21.0	19.0	17.0	13.0	11.5	9.0	6.5
	3	28.0	26.0	23.0	18.0	16.0	13.0	10.0
	4	41.0	38.0	34.0	24.0	21.0	21.0	17.0
	5	53.0	49.0	45.0	36.0	34.0	28.0	23.0

NOTES

(a) If existing concrete pavement has very few cracks, indicated thickness of overlay is reduced by 4″ but to final thickness of not less than 2″.

(b) If base or special subgrade is beneath existing pavement, total overlay is reduced by this amount but not to less than 2″ total.

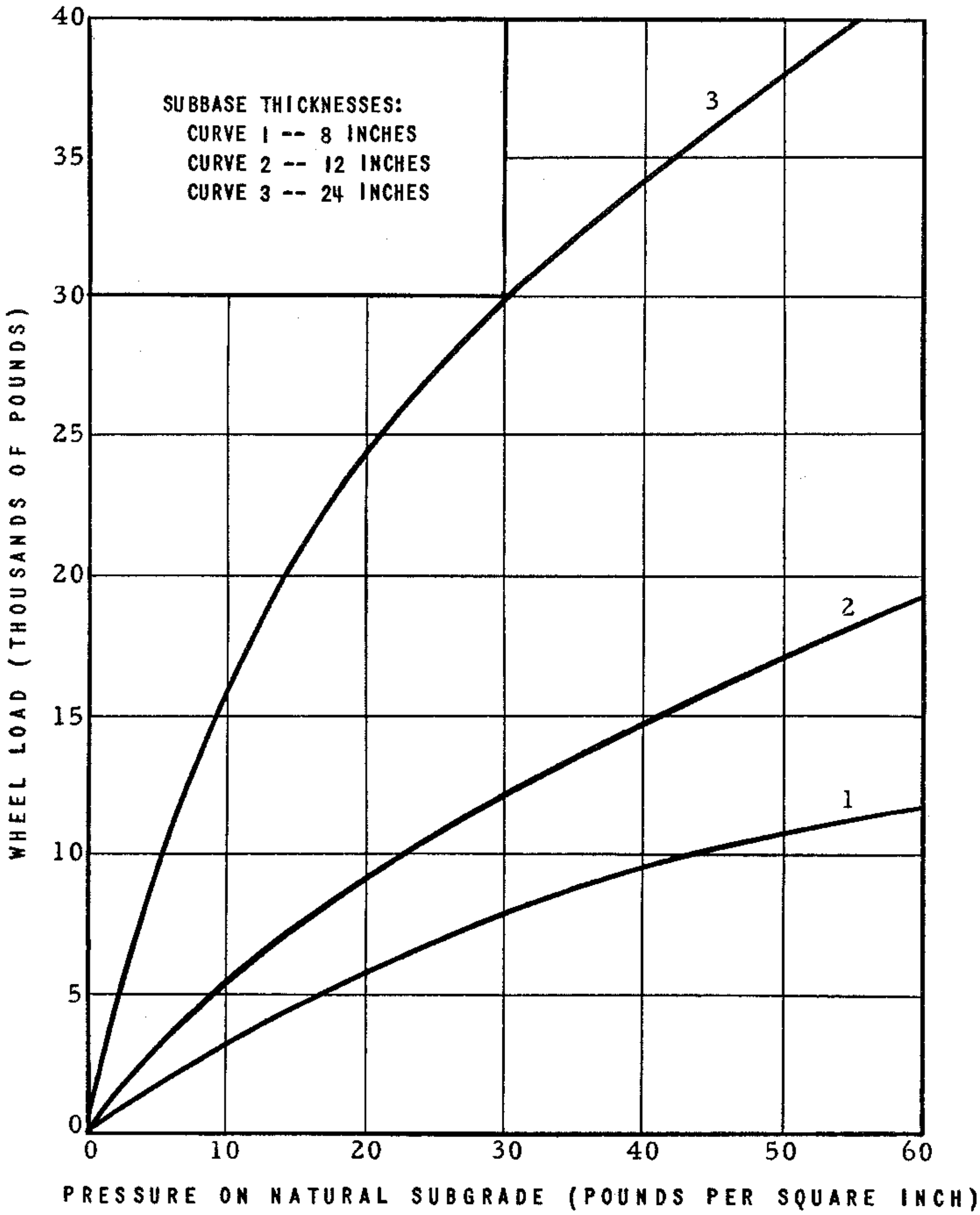
(c) Bituminous concrete pavement thickness varies with magnitude of traffic and load. For class 1 and 2 roads, 3½″ of hot mix is recommended with 35,000- and 40,000-pound axle load; 3″ with 25,000 and 30,000; and 2½″ with 15,000 and 20,000. For roads with lower traffic, thickness is reduced in same proportion to not less than 2″.

(d) If base is 3″ or less, a penetration base is recommended. If base is 3″ to 6″ in depth, a 1½″ well-graded, mechanically stabilized base is recommended.

(e) For overlays greater than 8½″ in thickness, a 6″ mechanically stabilized well-graded base is recommended. Below 8½″, material may be select subgrade of lower quality (but with adequate C.B.R. for the location).

Chart XIV

RELATIONSHIPS BETWEEN WHEEL LOAD AND PRESSURE ON NATURAL SUBGRADE, FOR SELECTED SUBBASE THICKNESSES*



* Plate Diameter: 12 inches
Strength Index: Base Type Gravel = 100 percent
K = 82 psi/inch

Source: *Proceedings*, Highway Research Board, Vol. 32 (1952), p. 117.

subgrade support to thicknesses of special subbase applied over natural subgrade.

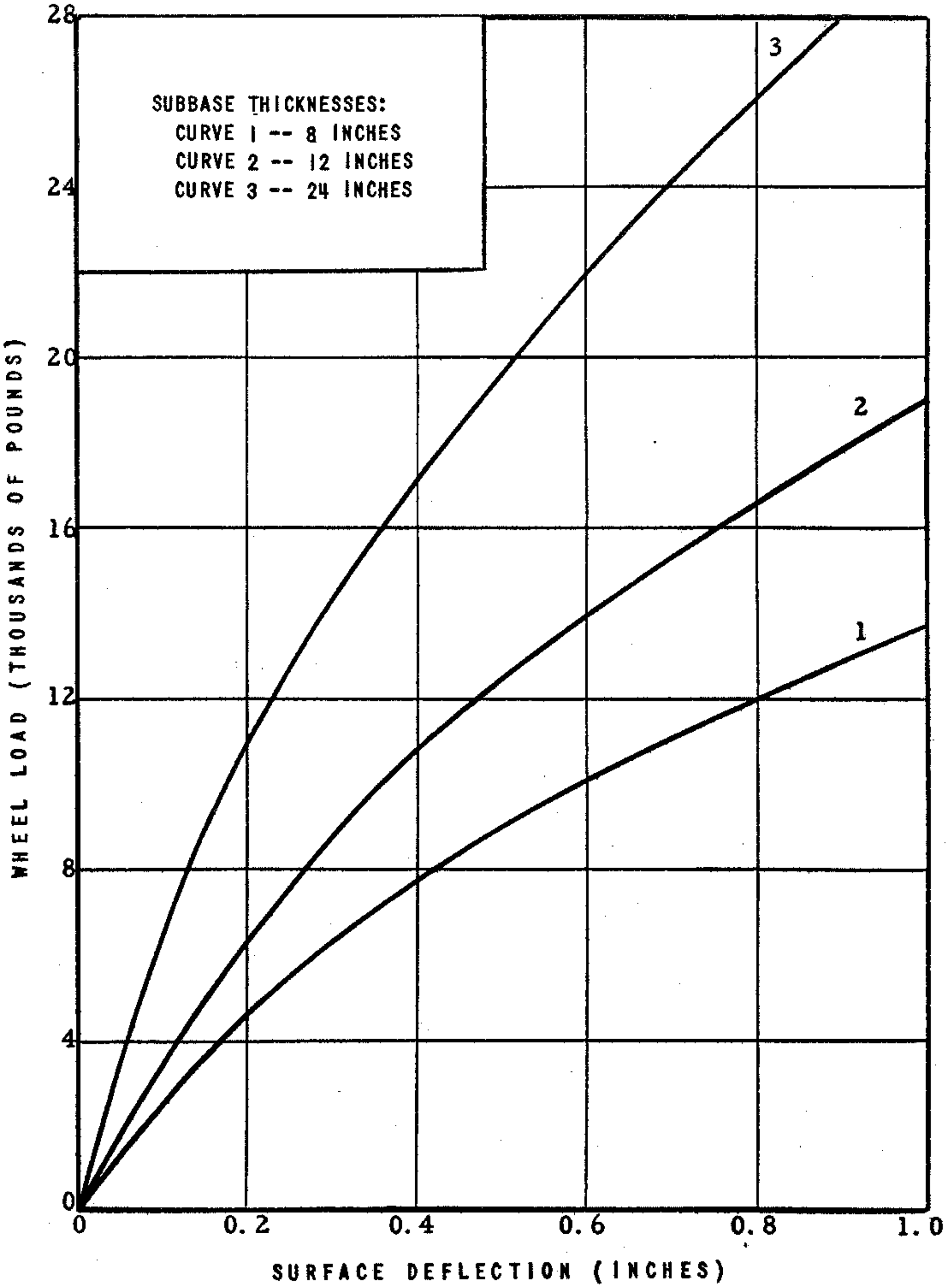
Chart XIV should be read as follows: A maximum pressure on a natural subgrade of 10 psi will be generated by a wheel load of 3,125 pounds if special subbase thickness is 8 inches; by a wheel load of 5,500 pounds if special subbase thickness is 12 inches; and by a wheel load of 15,800 pounds if thickness is 24 inches. Since the stress and deflections of the subgrade are indicated as equal under all three loadings, the life for flexible pavement would presumably be equal.

Chart XV shows: 0.2 inch of surface deflection is produced by a wheel load of 4,600 pounds if thickness of special subbase is 8 inches; by a wheel load of 6,400 pounds if thickness is 12 inches; and by a wheel load of 10,800 pounds if thickness is 24 inches. These increases in weights accompany increases in thickness even though the natural subgrade may have very poor support characteristics.

While it appears that pavement life, thickness, and subgrade relationships can be established most advantageously from actual pavement performance data, this does not preclude use of laboratory studies to serve as checks and supplements to field studies.

Chart XV

RELATIONSHIPS BETWEEN WHEEL LOAD AND SURFACE DEFLECTION FOR SELECTED SUBBASE THICKNESSES*



* Plate Diameter: 12 inches
Strength Index: Base Type Gravel = 100 percent
K = 82 psi/inch

Source: *Proceedings*, Highway Research Board, Vol. 31 (1951), pp. 110, 111

B. Rigid Pavements

A concrete slab resting on a subgrade serves as a thick bearing plate, and a mathematical method may be used to determine the proper depth of slab after the width and length have been established.[2]

Optimum thickness of rigid concrete pavements for specified wheel loads and particular climatic and subgrade conditions has been studied by engineers and mathematicians for many years, and in some cases results have been subjected to limited field tests. The validity of specific assumptions on which theoretical equations are based has not, as yet, been established by experiments (the most notable of which have been those conducted by the U. S. Bureau of Public Roads and the Highway Research Board). Results of experiments have been correlated with theoretical values and adjustments of theoretical equations made, so that semiempirical solutions are now available which may be used for design or investigation. Semiempirical design methods are required, since many indeterminate equations enter the problem.

The thickness of a concrete pavement slab is best based on an examination of the actual repetitions of the longitudinal-edge stress resulting from restrained temperature warping and applied loads.[3] Wheel loads are used, and dual tires are considered as one wheel. If the load is applied in the daytime, Westergaard's equation is recommended for this maximum load stress (parallel to the edge and producing transverse cracks) at a point along the free edge (due to a load applied at this point). Bradbury (B-4), by using 0.15 for Poisson's ratio for concrete, has shown that Westergaard's equation reduces to this form:[4]

$$\sigma_{ex} = 0.57185 \frac{P}{d^2} \left[4 \log_{10} \frac{l}{b} + 0.3593 \right] \quad \text{(I)}$$

To this load stress must be added the restrained temperature warping stress (which can be three times as great during parts of the day as at night [B-31]). The maximum unit stress along the edge of the slab due to this restrained warping may be found from the equation:

$$S_{ex} = \frac{C_x E e \Delta t}{2} \quad \text{(II)}$$

where: C_x is a coefficient (evaluated by Bradbury)

If the load is to be applied at night, the maximum load stress along (and parallel to) the edge of the pavement, for a load at this point, will be larger than that resulting from Equation I, since the slab is curled upward. The resulting load stress is defined by Kelly's equation (B-14), a modification of Equation I:

$$\sigma_{ex} = 0.57185 \frac{P}{d^2} \left[4 \log_{10} \frac{l}{b} + \log_{10} b \right] \quad \text{(III)}$$

The maximum *combined* unit stress will result from use of values from Equation I together with the restrained warping stresses present during the day. (The stresses due to restrained warping are opposite in sign to the load stresses at night, and the curling stresses at night are only about 1/3 of their value in the daylight hours.) Chart XVI shows the type of stress curves which are obtained when the load stress and restrained temperature warping stress values are combined.

An attempt was made to evaluate the maximum unit stress in the bottom fibers of the interior portion of the slab when restrained temperature warping stress at that point is combined with maximum load stress (due to a load applied directly above that point). Theoretical equations and empirical equations available to treat this problem

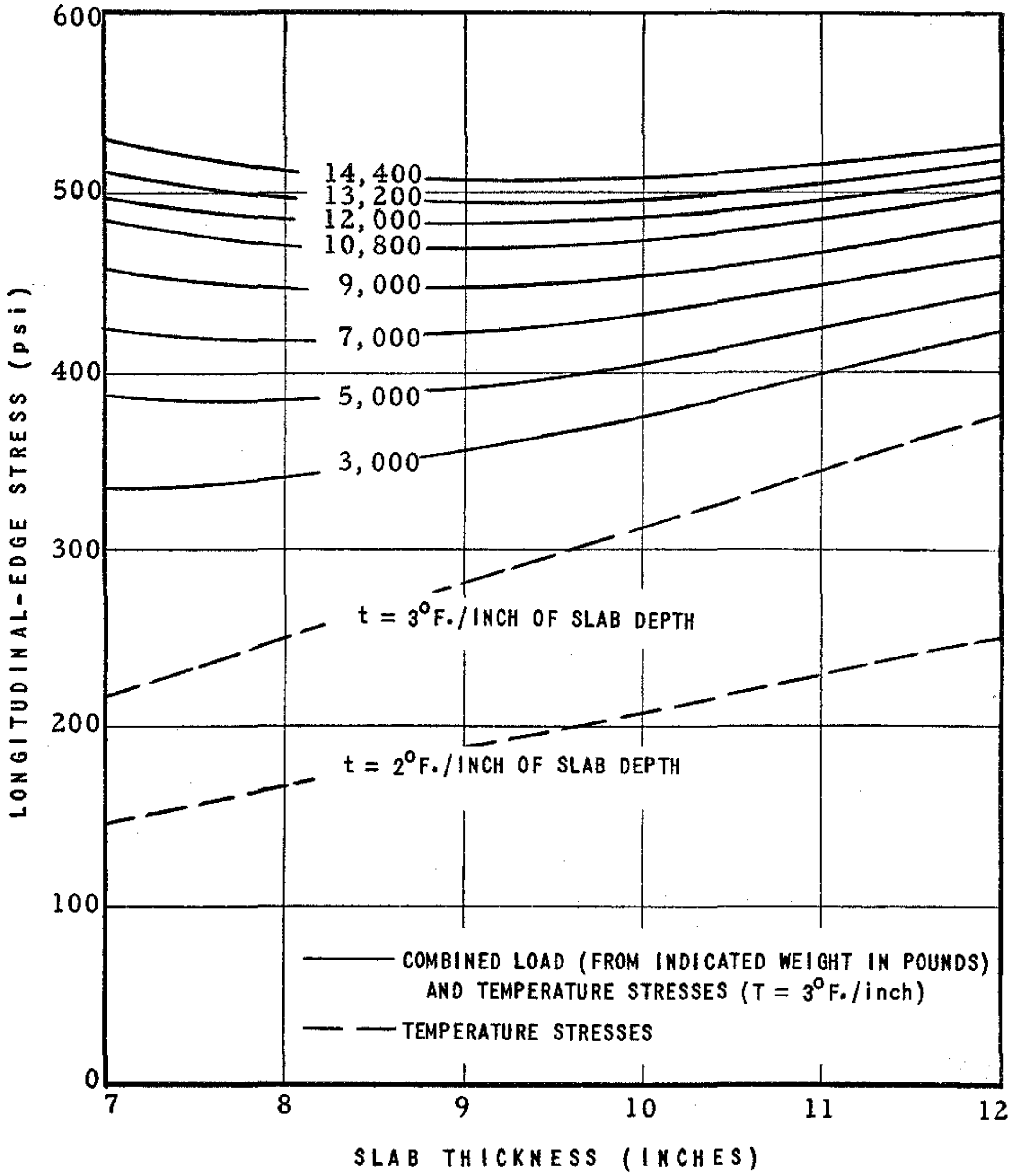
[2] Mathematical design procedure indicates that for design purposes slab lengths of about 30 feet are equivalent to infinite slab lengths. The length now used in Pennsylvania is 61½ feet.

[3] For history of analyses of pavement design, see Appendix B.

[4] A complete list of symbols is presented on page 91. Details of equations are given in Appendix C.

Chart XVI

RESTRAINED TEMPERATURE WARPING STRESSES AND COMBINED LOAD AND TEMPERATURE STRESSES: LONGITUDINAL EDGE*



* Slab Length ≧ 45 Feet; K = 200 psi/inch

are those of Westergaard (B-39) as modified to match the measured stresses of the Arlington road tests. However, these equations contain two variables, L and Z, which depend on interrelated slab, soil, and load values and can be determined only by field measurements of stress and deflection. At the present time, available measurements are insufficient for accurate estimates of L and Z for various combinations of slab, soil, and load conditions. Bradbury suggested that, since each of the variables (L and Z) had certain known limits, an average of these limiting values could be used to give fair results. However, use of Bradbury's suggestion will not result in correct values, since the means of the two variables do not even approximately match the values found in the Arlington road tests, and the stress σ_{ix} is appreciably affected by L and Z. Since the solution for σ_{ix} could not be applied to obtain usable results, and since observations of pavement cracks on the highways of Pennsylvania appeared to show that transverse cracks began at the edge and not in the interior of the slab, no further attention was given this problem.

The other position of loading examined in detail was that of corner loading to produce cracking across the corner. A study of practical applications of theoretical equations indicates that the maximum load stress at the plane of weakness when a load is placed at the outside corner of a slab can be found by use of one of Pickett's equations (B-18):

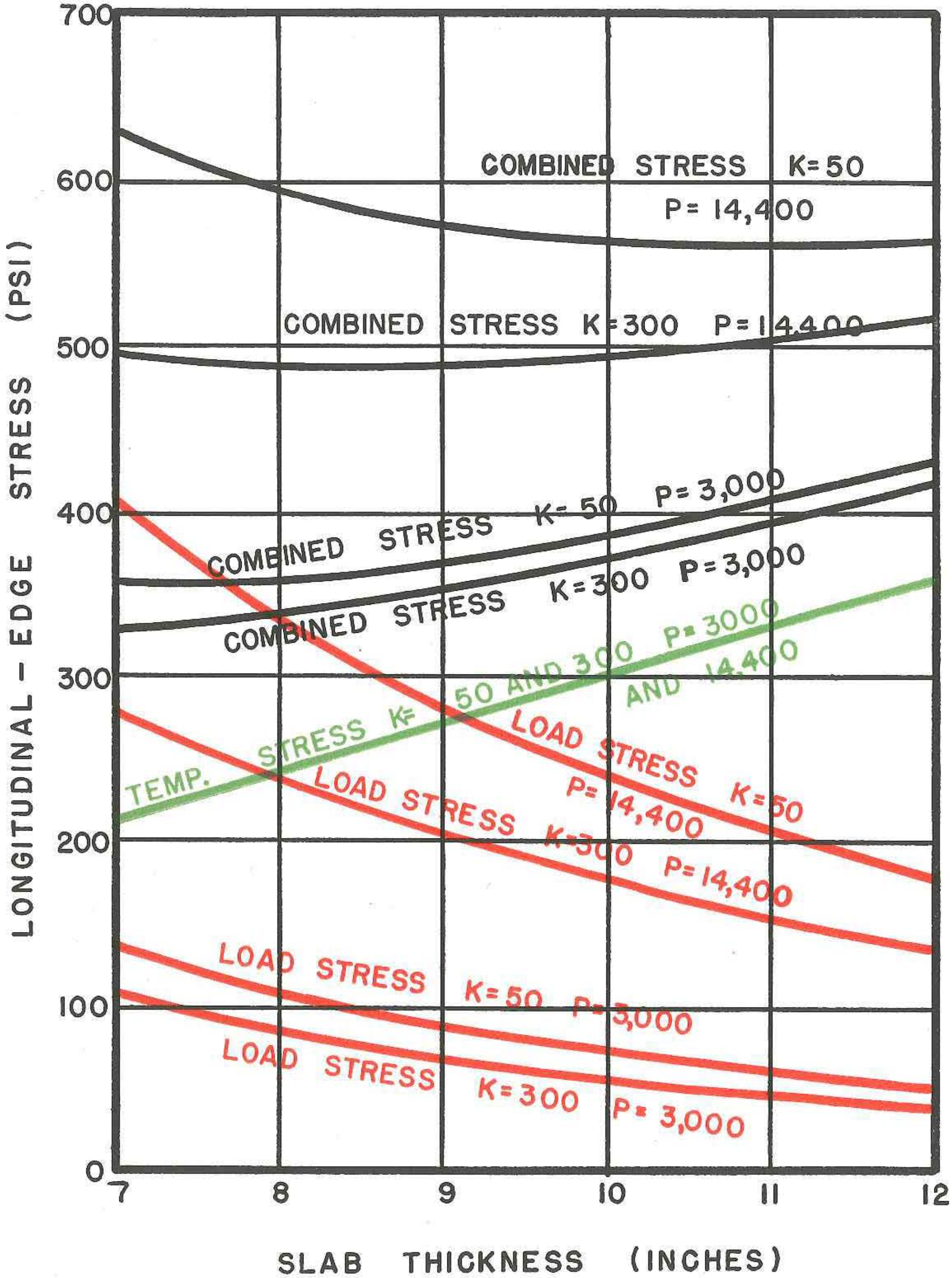
$$\sigma_c = \frac{4.2P}{d^2}\left[1 - \frac{\sqrt{a_1/l}}{1.1 + 0.185\, a_1/l}\right] \qquad \text{(IV)}$$

The length of slab affects the restrained temperature warping stresses in a slab and therefore considerably affects the total longitudinal-edge stresses; however, since the critical section with a load at the corner is close to the corner, there is practically no temperature warping stress at such a section. The combined (load and temperature) edge stress is much larger than the corner stress; and since it is known that only stresses which exceed 50-55 percent of the flexural modulus of rupture of the concrete contribute to the development of cracks, 9-inch Portland cement concrete pavements placed on very good soils (k of 300 psi/in or higher) may develop numerous transverse cracks and yet rarely crack across the corner. The effect of restrained temperature warping is such that there is nothing to be gained from increasing the depth of the slab, if its length is greater than 30 feet, since any decrease in load stress is more than offset by the increased temperature warping stress at a section near the mid-length of the slab. When slabs less than 15 feet long are placed on very poor soils (k under 100 psi/in), the corner stress may be as great as the total longitudinal-edge stress, but, since it is impractical to use such short slabs, there appears to be little justification for use of a practical highway design method based on the corner crack.

Chart XVII shows comparisons of maximum longitudinal edge stresses under two different wheel loads, on good and poor subgrades, and with lengths of slabs exceeding 45 feet.

Along with study of maximum stresses due to restrained temperature warping and to corner, longitudinal-edge, and interior placement of loads, the *frequency* of the maximum stresses must be considered. Since, as stated before, there appears to be no practical and accurate method of computing the maximum load stresses due to interior placement of loads, that loading has been eliminated from the study. The transverse distribution of vehicles across the roadway will result in equal numbers of repetitions of load stresses at the corner and at the longitudinal edge. However, temperature warping stresses are additive to the longitudinal-edge load stresses only during the day; other studies have revealed that the maximum temperature differential in the pavement slab oc-

Chart XVII
MAXIMUM LONGITUDINAL-EDGE STRESSES IN PAVEMENT SLABS, ON GOOD AND POOR SUBGRADES, FOR SPECIFIED WHEEL LOADS*



* Slab Width = 12 Feet
Slab Length ≥ 45 Feet
$E = 4{,}000{,}000$ psi
$\mu = 0.15$
$e = 0.000005$

curs during only about $\frac{1}{2}$ of the year and that during the other half of the year $\frac{2}{3}$ of the maximum temperature differential occurs. Therefore, it appears that only about $\frac{1}{8}$ of the traffic passes over the lane at a time when the load stress will be additive to the maximum temperature warping stress at the longitudinal edge, and about $\frac{1}{8}$ of the traffic also passes when $\frac{2}{3}$ of the maximum temperature warping stress is effective (B-4). One might assume that the larger number of repetitions of the corner stress would cause failure of the slab, but only those stresses above 50-55 percent of the flexural modulus of rupture (approximately 350 psi) harm the pavement. Stresses due to corner loads will seldom approach this harmful range.

If the design values used in equations do not match field conditions, correct results cannot be obtained. Similarly, assumptions upon which theoretical equations are based must represent actual conditions. Although the factor *k*, used in Equation I, may vary for the soil under a slab (or may decrease at a point as loads are repeated at that point [B-4]), the equation may be successfully employed if a minimal value is used. However, should any part of a subgrade or base differ so much from a neighboring part that slab heaving or settlement occurs, the slab will not be uniformly supported at all points but will have voids beneath it. These voids where *k* approaches zero lead to load stresses in the slab approximately double those of the case when *k* is 200 psi/in, and the slab might fail quickly. Therefore, a Portland cement concrete pavement should not be placed on a subgrade or base which has tendencies to shift, heave, or "pump," and the design procedures recommended in this report do not cover design of Portland cement concrete pavement under these conditions. Poor subgrade conditions should first be corrected by drainage and use of granular, Portland cement, or bituminous admixtures. Some subgrade material cannot be treated and drained satisfactorily, and must be completely discarded.

If the bearing capacity of the natural subgrade is so low that a desired pavement life cannot be obtained by using the optimum pavement thickness, special subgrades should be used or the natural subgrade should be mechanically or chemically treated.

For practical design, the only way to determine what must be done to a natural subgrade to improve its bearing capacity is to make special trial subgrade thicknesses or types of treatment in the field and to conduct plate bearing tests to determine increases in subgrade modulus, *k*.

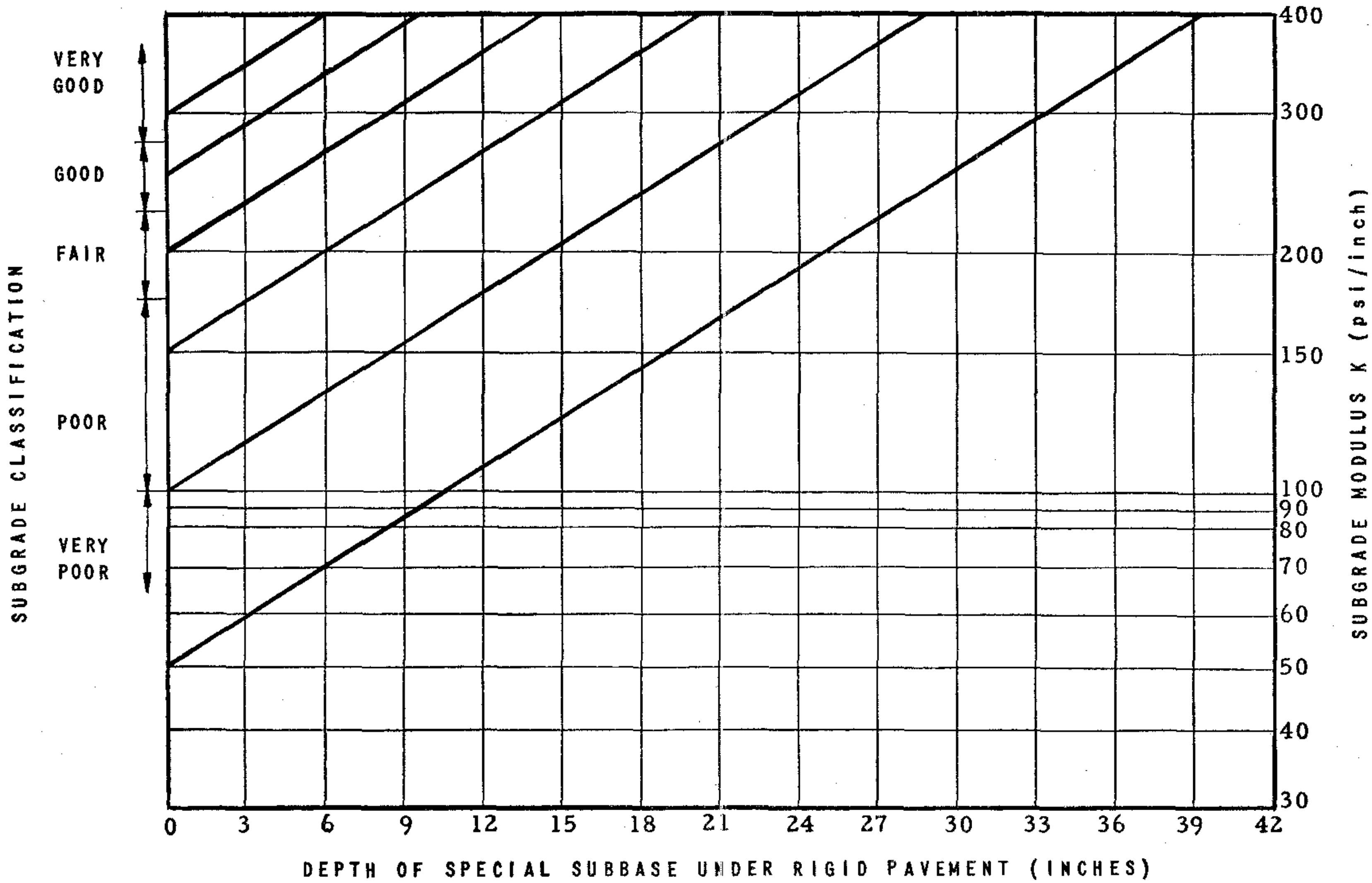
The data shown in Chart XVIII demonstrate the improvement of subgrade support. The curves relate initial natural subgrade modulus and depth of special subbase material to the resultant subgrade modulus under the pavement. These curves represent subgrades with initial *k* values of 50, 100, 150, 200, 250, and 300. The depth of special subbase material necessary to improve initial subgrade to higher support values can be read directly from the chart. These curves, based largely on theory, compare favorably with those for similar subgrade improvements under flexible pavements and, subject to later field study and modification, appear to have considerable merit as provisional bases for design estimates.

After the quality of the subgrade has been improved to the point at which it will not collect and hold water, a subbase or special subgrade course of well-graded stone or gravel should be placed to a depth of at least twelve inches if axle loads are above 15,000 pounds. A six-inch depth of special subgrade is recommended if axle loads are less than 15,000 pounds. Through this well-compacted special subgrade (carried from ditch to ditch), the water which would find its way through joints would be drained from the roadbed, and the possibility of frost action and pumping would be greatly reduced.

Comprehensive soil tests and soil studies (including geological origin) should be employed to determine subgrade treatment necessary to im-

Chart XVIII

RELATIONSHIPS BETWEEN SUBGRADE SUPPORT AND DEPTH OF SPECIAL SUBBASE



Source: *Proceedings*, Highway Research Board, Vol. 32 (1952), p. 120.

Table 10

PERCENT DISTRIBUTION OF DAYTIME TRANSVERSE PLACEMENT OF COMMERCIAL VEHICLES ON PAVEMENT LANES 18, 20, 22, AND 24 FEET WIDE*

Transverse Placement of Left Wheels to Right of Center Line†		*Pavement Lane Width and Commercial Vehicle Traffic Passage Condition*											
		18 Feet			*20 Feet*			*22 Feet*			*24 Feet*		
Range (feet)	*Average (feet)*	*Free Moving*	*Meeting Pass-enger Cars*	*All Other than Passing*	*Free Moving*	*Meeting Pass-enger Cars*	*All Other than Passing*	*Free Moving*	*Meeting Pass-enger Cars*	*All Other than Passing*	*Free Moving*	*Meeting Pass-enger Cars*	*All Other than Passing*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
—2.9 to —2..	—2.5	0.7%	0.7%	0.5%	0.1%	0.3%	0.3%	...	...	...	0.6%	...	0.3%
—1.9 to —1..	—1.5	0.8	0.7	0.5	0.6	0.5	0.7	1.4%	...	1.5%	0.6	...	0.5
— .9 to 0....	—0.5	2.6	2.5	2.5	4.3	.4	2.7	5.3	...	4.9	2.3	...	1.2
0 to .9......	0.5	29.9	6.8	24.3	17.2	3.2	14.1	7.9	1.4%	7.5	4.6	1.2%	3.1
1 to 1.9......	1.5	28.5	54.1	49.1	42.2	27.2	37.7	22.8	19.0	21.1	33.7	21.6	27.4
2 to 2.9......	2.5	16.7	27.8	21.2	30.8	46.4	35.9	47.7	40.1	47.3	37.1	37.1	39.4
3 to 3.9......	3.5	0.8	7.4	1.8	3.8	20.6	7.7	13.5	23.8	15.4	17.1	21.6	20.3
4 to 4.9......	4.5	...	...	...	0.8	1.4	0.9	1.4	15.7	2.3	3.4	17.0	6.9
5 to 5.9......	5.5	...	...	0.1	0.2	...	‡	...	...	...	0.6	1.5	0.9
6 to 6.9......	6.5	...	...	...	‡	...	‡	...	...	...	...	...	...
Total		100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0	100.0%	100.0%
Average (feet)		1.3	1.8	1.4	1.6	2.3	1.8	2.0	2.8	2.1	2.2	2.9	2.5

*Two-lane pavements with grass or gravel shoulders at least 4 feet in width.

†Negative values indicate wheels to left of center line.

‡Less than 0.05 percent.

Table 11

LATERAL DISTRIBUTION OF COMMERCIAL VEHICLES*

Transverse Placement of Right Wheels to Left of Right Edge of Pavement	*Pavement Lane Width*			
	9 Feet	*10 Feet*	*11 Feet*	*12 Feet*
(1)	(2)	(3)	(4)	(5)
½ foot	15.7%	5.8%	1.6%	0.7%
1 foot	17.7	11.7	3.8	1.8
1½ feet	23.6	17.5	8.4	3.7
2 feet	18.6	18.7	16.6	6.6
2½ feet	12.5	18.6	22.9	10.6
3 feet	6.7	14.2	17.9	15.4

*Right wheels to left of right edge of pavement, only.

prove drainage characteristics and load-bearing values and to eliminate shifting under loads. A pavement placed on a poor subgrade cracks quickly, and it is impossible to keep a pavement of any type smooth on a poor subgrade.

Determinants of pavement design are subgrade support, and magnitude and frequency of stresses in slabs of specific dimensions. Magnitude and frequency of load stresses are in part determined by the lateral distribution of wheel loads on the pavement. Although the maximum load stress in the bottom fibers at the free longitudinal edge of a concrete slab occurs when the load is placed over that point, the amount of the stress at this point must also be considered when a load passes within 1, 1½, 2, or 2½ feet of the outer edge of the slab. The magnitude of such load stress can be determined by application of the findings of the Arlington road tests and of Road Test One-Md. (Throughout this report, the term "longitudinal-edge stress" refers to the stress parallel to the free longitudinal edge at a point six inches from the edge).

From extensive speed-placement studies conducted by the Public Roads Administration (Bureau of Public Roads) (B-28) the transverse placement of passenger cars and commercial vehicles on highways with 9-, 10-, 11-, and 12-foot lanes has been determined. Extracts of these data for commercial vehicles only are reproduced in Table 10, and the same data appear in Table 11 in a form more suited to this study. (Since wheel loads less than 3,000 pounds produce negligible load stresses in pavements, only commercial vehicles are considered.) The percentages of commercial traffic at 1, 1½, 2, 2½, 3 feet, etc., from the outer edge of the pavement (and their longitudinal edge stress effects referred to in the preceding paragraph) are taken into account in determining the numbers of repetitions of stresses at the edge of the pavement.

A method was devised to combine the effects of the many different stress values obtained; this method was based on Bradbury's work and his fatigue curve of concrete in flexure. The method may be illustrated as follows: If a 5,000-pound load at the edge produces a combined edge stress of magnitude A and a 5,000-pound load one foot from the edge produces a combined edge stress of magnitude B, and it is found from the fatigue curve of concrete that L repetitions of stress A will cause failure of the concrete and that M repetitions of stress B will cause similar failure, a single repetition of the latter loading is equivalent to L/M repetitions of the former loading. For example, if twenty 7,000-pound loads passed successively along the edge and each produced a combined edge stress of magnitude C (R number of which would cause failure), they would be the equivalent of 20 L/R repetitions of a single 5,000-pound load placed at the pavement edge. In this way any traffic pattern can be reduced to a number of equivalent effects of a basic load placed at a single position on the pavement slab. A 5,000-pound edge load was used in this study.

The total number of effective 5,000-pound wheel loads for any load group may be expressed as a percentage of the total number of loads in that group (equivalent wheel-load percent factor). These factors may be computed for different soil bearing values, slab depths, and slab widths. Table 12 lists the equivalent wheel-load percent factors for slabs 12 feet wide and for common soil values and depths of slab. Equivalent wheel-load percent factors show a well-defined relationship between 12-foot and 11-, 10-, and 9-foot slab widths; for 11-foot lanes, factors are 2.25 times those of a 12-foot lane; for 10-foot lanes, 8.1 times those of a 12-foot lane; and for 9-foot lanes, 18.1 times those of a 12-foot lane. Analysis may be made for pavements with 12-foot lanes and these conversions applied to estimate life of pavements of other widths.

After the effects of the different wheel loads are expressed in terms of 5,000-pound wheel loads placed at the longitudinal free edge of the pavement slab, the estimated fatigue points contained in Chart XIX are used to determine the time in years before the first crack will appear. In the design or study of a pavement, a factor relating this number of years to operating life, in years, is needed.

The method of design explained above was applied to existing concrete highways in Pennsylvania for which traffic data, some soils information, and existing pavement condition were known. Results indicated that a pavement placed on a soil with k of 250 psi/in or more would have an operating life about 2 times the number of years to first cracking, while similar pavements on subgrades with k less than 250 psi/in would have an operating life of 4/3 the number of years to first cracking. These factors, 2 and 4/3, are termed life factors.

It appears that the volume and composition of traffic on our highways will continue to change. Average daily traffic for 1951 was chosen as a base for traffic estimation in this method of design. Chart VIII contains a curve relating 1951 traffic volumes to estimated future traffic volumes. It will be noted that the curve of Chart VIII is extended for years prior to 1951, in order to compute operating life in years for existing pavements. If traffic estimates for a year other than 1951 (e.g., 1954) are used in design, a new traffic prediction curve is plotted with that year (1954) as the base.

From the foregoing, it may be observed that, for specific subgrade support and lateral placement of vehicles, the optimum thickness of a highway depends upon the *relative* distribution of wheel load weights, while the operating life of a highway is the total *number* of axle loads (in the ratios expressed in the relative distribution) which will cause complete destruction.

The following data are required to determine thickness of Portland cement concrete pavements:

1. Relative distribution — percent — of wheel load weights (traffic intensity)
2. Modulus of subgrade reaction, k, determined from field plate bearing tests conducted when the subgrade is in the worst condition. Since 12″ of special subgrade is recommended beneath any Portland cement concrete pavement,[5] these plate bearing tests may be conducted with special subgrade material in place. (California Bearing Ratio tests may be substituted for plate bearing tests and equivalent k values obtained.)

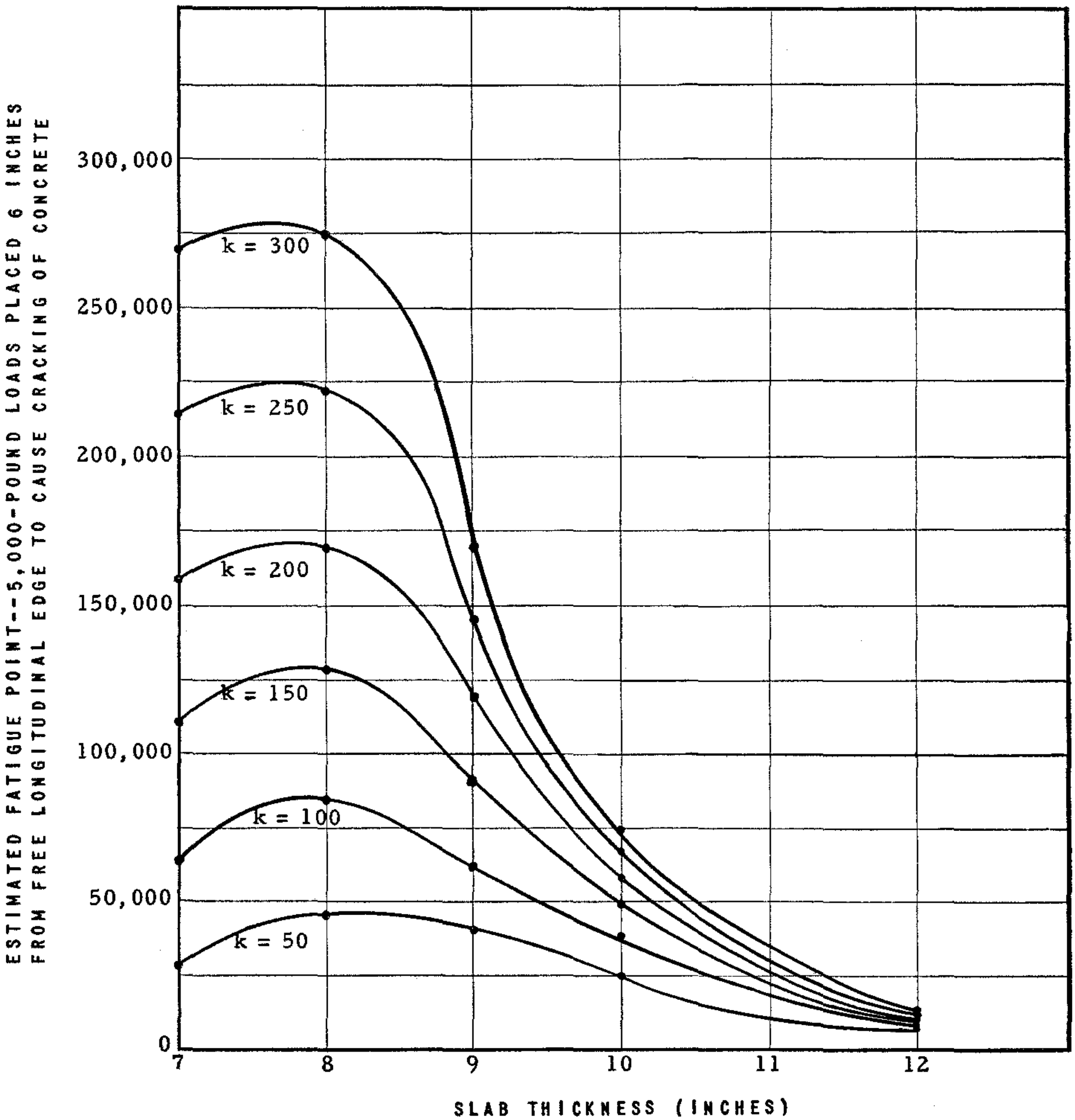
[5] Only 6″ of special subgrade is recommended if heaviest axle load is 15,000 pounds or less.

Table 12
Equivalent Wheel Load Percent Factors for Twelve-Foot Slabs at Specified Subgrade Moduli and Pavement Thicknesses

Wheel Load (Pounds)	*Subgrade Modulus and Reinforced Concrete Pavement Thickness*																			
	k = less than 100				*k = 100-175*				*k = 175-225*				*k = 225-275*				*k = greater than 275*			
	7″	8″	9″	10″	7″	8″	9″	10″	7″	8″	9″	10″	7″	8″	9″	10″	7″	8″	9″	10″
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
3,000....	0	0.01	0.01	0.03	0.02	0.01	0.01	0.05	0	0	0.01	0.04	0.02	0.01	0.01	0.05	0	0	0	0.04
4,000....	0.06	0.07	0.09	0.11	0.04	0.04	0.06	0.09	0.03	0.05	0.05	0.09	0.04	0.03	0.05	0.09	0.04	0.04	0.06	0.10
5,000....	0.12	0.13	0.18	0.22	0.11	0.12	0.17	0.27	0.09	0.13	0.16	0.24	0.07	0.09	0.15	0.25	0.09	0.09	0.15	0.26
6,000....	0.50	0.39	0.43	0.42	0.31	0.33	0.35	0.49	0.39	0.39	0.40	0.51	0.27	0.29	0.37	0.48	0.26	0.24	0.36	0.52
7,000....	1.30	0.98	0.90	0.84	0.86	0.77	0.70	0.85	0.95	0.77	0.79	0.97	0.72	0.66	0.73	0.83	0.62	0.58	0.74	0.78
8,000....	3.2	2.1	2.0	1.5	2.4	1.8	1.4	1.4	2.4	1.5	1.1	1.3	1.8	1.3	1.4	1.3	1.3	1.1	1.3	1.2
9,000....	8.7	5.5	3.9	2.9	5.1	3.6	2.6	2.3	5.1	2.9	2.3	1.9	3.8	2.6	2.3	2.0	2.5	2.1	2.3	1.9
10,000....	22.4	12.3	7.4	5.0	10.2	6.9	4.6	3.9	9.2	5.2	3.8	3.0	6.8	4.5	3.8	3.0	4.5	3.7	3.8	2.9
11,000....	40.6	25.9	13.2	7.8	18.4	11.3	7.5	5.8	15.2	8.2	5.9	4.6	11.4	7.3	5.8	4.3	7.9	6.4	5.7	3.9
12,000....	50.4	34.7	23.4	13.0	31.8	19.2	11.8	8.1	25.6	13.6	9.3	5.8	19.1	11.7	9.0	5.8	12.8	10.0	8.3	5.4
13,000....	78.6	47.7	38.6	20.8	54.1	31.1	19.2	12.9	48.6	23.6	17.2	9.7	34.3	19.4	14.7	9.4	19.6	14.8	11.9	7.7
15,000....	199.0	111.0	80.0	42.1	148.0	71.5	41.2	28.1	138.0	63.3	39.8	23.1	91.9	48.1	32.9	18.4	45.5	32.9	25.8	13.7
17,500....	468.0	240.0	149.0	71.7	220.0	142.0	88.0	48.0	220.0	117.0	61.6	31.6	140.0	88.0	52.0	26.0	77.5	60.7	41.5	21.2
20,000....	2880.0	402.0	281.0	136.0	370.0	268.0	162.0	75.0	322.0	252.0	101.0	54.5	230.0	160.0	87.0	44.0	140.0	105.0	68.1	33.0

Chart XIX

RELATIONSHIP OF CONCRETE SLAB THICKNESS TO ESTIMATED FATIGUE POINT*



* Modulus of rupture = 700 psi.

For each wheel-load group, the traffic-intensity percent distribution is multiplied by equivalent wheel-load percent factors—contained in Table 12—(taking into account the proper subgrade modulus k), to yield number of equivalent 5,000-pound wheel loads. This computation is made for pavement thicknesses of 7, 8, 9, and 10 inches, and, for each of these thickness groups, the sum of equivalent 5,000-pound wheel loads for the traffic-intensity distribution is obtained. These sums indicate the effect on each of the pavements of the traffic intensity (distributed normally across a traffic lane).

The total number of repetitions of 5,000-pound wheel loads to the estimated fatigue point are shown in Chart XIX for specified subgrade moduli and thicknesses of Portland cement concrete pavement. The total number of repetitions of the 5,000-pound wheel load (from Chart XIX) is divided by the sum of 5,000-pound load equivalents from the traffic-intensity distribution, and the quotient obtained is the total number of commercial axle passages to the estimated fatigue point.

Optimum thickness of a pavement is that which permits the greatest number of wheel-load passages, and this maximum number of passages may be noted after divisions similar to that above are made for each of the pavement thicknesses considered.

To estimate life of a pavement, in years, at *present* traffic volumes, the following information is needed:

1. Average daily traffic
2. Estimated percent of commercial vehicles
3. Estimated number of axles per commercial vehicle (A tandem unit is counted as two axles.)
4. Distribution of commercial traffic between lanes (For two-lane roads, half the commercial traffic is generally assumed for each lane.)

In order to estimate life, the number of commercial axles per lane per year is first obtained as follows: Average daily traffic is multiplied by the number of days per year, by the estimated percentage of commercial vehicles, and by the estimated number of axles per commercial vehicle, and the product is divided by the number of lanes (usually two). The number of commercial axles per lane per year is then divided into total number of commercial axle passages to estimated fatigue point to provide a quotient showing number of years to estimated fatigue point at present traffic volumes. Since the operating life of the highway is greater than the number of years to the estimated fatigue point (initial cracking of a slab does not make a highway unusable), estimated life to fatigue point is multiplied by a life factor (2, in the case of pavements constructed on subgrades with k equal to or greater than 250 psi/in., and $4/3$ for subgrades with k less than 250 psi/in.) The product of this multiplication, estimated operating life at *present* traffic volumes, may be converted to operating life at anticipated traffic volumes by reference to Chart VIII (or charts similar to VIII for other estimates of future traffic).

Traffic data which are usually available are estimates of average daily traffic, percentage of commercial vehicles, and commercial traffic intensity —expressed as a percentage of axles of commercial vehicles in various load categories. Of these three items, commercial traffic intensity is the most difficult to obtain, but accurate design is impossible without it. Also, before any attempt can be made to compute pavement life, the number of commercial axles using the pavement must be estimated from the average daily traffic by use of axle factors per commercial vehicle. Recommended (on the basis of Pennsylvania loadometer station data) are 2.60, 2.40, and 2.20 for heavy, medium, and light truck traffic, respectively.

Examples of estimated commercial traffic-intensity distributions for fifteen loadometer stations in Pennsylvania appear in Table 13. Optimum thicknesses for these fifteen sample distributions and for the five classifications of subgrade soil support appear in Table 14, together with pavement life estimated on the basis of 10,000 commercial-axle passages per 12-foot lane per day. It should be noted that the thickness of pavement shown in Table 14 is that recommended for longest pavement life (in terms of relative distribution of wheel weights), regardless of actual number of commercial-axle passages per day.

For traffic intensities of Table 13, and any classification of subgrade soil, pavement life for 10,000 commercial-axle passages per lane per day and optimum thickness of pavement can be read directly from Table 14. If a number of commercial-axle passages other than 10,000 is used for estimating purposes, the reciprocal ratio of the number of commercial-axle passages to 10,000 may be used to convert to life in years for the estimated number of axle passages. The life factor and future traffic estimates may then be employed.

Lanes of 9-, 10-, or 11-foot widths receive more applications of edge loads from a specified traffic volume and traffic intensity than do 12-foot lanes (used in Table 14). Therefore, if lanes are not 12 feet wide, pavement life may be determined by dividing by the following:

For 11-foot lanes	2.25
For 10-foot lanes	8.10
For 9-foot lanes	18.00

This design method may be further illustrated with reference to the following subgrade and traffic information:

1. Subgrade modulus, $k = 200$ psi/in.
2. Average daily traffic = 7,700 vehicles
3. Percent commercial vehicles = 28 percent of average daily traffic
4. Number of axles per commercial vehicle = 2.60

   Where: tandem axle unit = 2 axles
5. Estimated commercial traffic intensity distribution = percent in Table 15, column (2).

For each wheel-load category, the commercial traffic intensity is multiplied by the equivalent wheel-load percent factor to obtain a product, equivalent 5,000-pound wheel loads. For each of the pavement thicknesses, the equivalent wheel loads are summed, and using Chart XIX to obtain total number of equivalent 5,000-pound wheel loads to estimated fatigue point, the following divisions are performed:

a. 7″ thickness: 158,000 ÷ .01597 = 9,900,000 commercial-axle passages

b. 8″ thickness: 170,000 ÷ .00957 = 17,800,000 commercial-axle passages

c. 9″ thickness: 120,000 ÷ .00760 = 15,800,000 commercial-axle passages

d. 10″ thickness: 59,000 ÷ .00720 = 8,200,000 commercial-axle passages

The greatest number of passages (with 1″ differences in pavement thickness) is shown for the 8″ pavement—17,800,000.

To proceed in estimating pavement life, the number of commercial-axle passages per lane per year is first determined from multiplication of average daily traffic (7,700) by percent commercial vehicles (28 percent), by number of axles per commercial vehicle (2.60), and by days per year (365), and dividing by number of lanes (2): the quotient is 1,023,000. Total number of passages (17,800,000) is divided by number of commercial-axle passages per year (1,023,000) and multiplied by life factor (4/3) to yield an estimated life of 23.2 years at *present* traffic volumes.

Table 13

ESTIMATED COMMERCIAL TRAFFIC INTENSITY DISTRIBUTIONS AT FIFTEEN LOADOMETER STATIONS

Wheel Load (Pounds)	*Loadometer Station 53*	*Loadometer Station 182*	*Loadometer Station 123*	*Loadometer Station 502*	*Loadometer Station 136*	*Loadometer Station 169*	*Loadometer Station 88*	*Loadometer Station 32*	*Loadometer Station 132*	*Loadometer Station 110*	*Loadometer Station 104*	*Loadometer Station 159*	*Loadometer Station 12*	*Loadometer Station 192*	*Loadometer Station 59*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
under 3,000.	36.4%	21.8%	28.2%	20.2%	21.1%	44.6%	43.3%	51.7%	32.8%	30.3%	41.0%	50.0%	49.2%	47.5%	55.7%
4,000	12.8	10.4	15.2	6.5	9.4	10.3	14.5	12.3	11.8	16.4	17.2	12.1	12.6	14.7	9.8
5,000	7.3	11.2	8.6	10.5	9.5	9.1	10.8	6.4	9.8	7.9	8.0	5.0	8.3	9.1	9.5
6,000	9.9	12.8	7.7	13.7	13.8	9.1	8.7	9.3	8.3	10.1	8.2	6.1	6.2	8.1	6.7
7,000	7.3	11.4	9.4	16.1	14.3	7.9	4.3	3.5	9.7	10.0	6.3	6.6	7.9	7.4	5.5
8,000	10.3	13.3	11.2	15.6	15.2	8.2	3.9	2.5	11.0	9.6	6.5	8.8	7.0	5.7	5.5
9,000	6.8	11.2	12.1	11.5	10.5	4.5	5.8	4.4	9.4	9.6	6.8	7.5	6.2	4.7	4.0
10,000	4.3	6.0	5.7	3.9	4.2	3.0	4.3	5.9	5.6	4.6	3.5	1.9	1.5	1.9	2.7
11,000	3.6	1.3	1.1	1.3	1.6	0.6	2.3	2.0	1.1	1.2	1.6	1.7	0.9	0.4	0.3
12,000	0.8	0.4	0.5	0.5	0.1	1.5	1.7	1.5	0.5	0.2	0.6	0	0.2	0.4	0.3
13,000	0.4	0.1	0.1	0.1	0.1	0.9	0.2	0.5	0	0.06	0.2	0	0	0.1	0
15,000	0.1	0.1	0.2	0.1	0.2	0.3	0.2	0	0	0.04	0.1	0.3	0	0	0
Totals	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%

Table 14

RECOMMENDED PAVEMENT THICKNESS AND PAVEMENT LIFE—ESTIMATED FOR 10,000 COMMERCIAL AXLES PER TWELVE-FOOT LANE PER DAY—FOR FIFTEEN SAMPLE COMMERCIAL TRAFFIC INTENSITIES AND FIVE CLASSES OF SUBGRADE SOIL SUPPORT*

Subgrade Soil Support Classification†	*Commercial Traffic Intensity*														
	Heavy					*Medium*							*Light*		
	Example 1 (Station 53)	*Example 2 (Station 182)*	*Example 3 (Station 123)*	*Example 4 (Station 502)*	*Example 5 (Station 136)*	*Example 6 (Station 169)*	*Example 7 (Station 88)*	*Example 8 (Station 32)*	*Example 9 (Station 132)*	*Example 10 (Station 110)*	*Example 11 (Station 104)*	*Example 12 (Station 159)*	*Example 13 (Station 12)*	*Example 14 (Station 192)*	*Example 15 (Station 59)*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Very Good..	8″ 16.8 yr.	8″ 17.1 yr.	8″ 17.2 yr.	8″ 17.2 yr.	8″ 17.4 yr.	8″ 18.9 yr.	8″ 18.9 yr.	8″ 20.3 yr.	8″ 20.6 yr.	8″ 22.1 yr.	8″ 23.6 yr.	8″ 25.4 yr.	8″ 36.5 yr.	8″ 38.8 yr.	8″ 42.6 yr.
Good	8″ 11.3 yr.	8″ 11.4 yr.	8″ 11.4 yr.	8″ 11.5 yr.	8″ 11.6 yr.	8″ 12.3 yr.	8″ 12.6 yr.	8″ 12.2 yr.	8″ 13.9 yr.	8″ 14.6 yr.	8″ 15.8 yr.	8″ 16.6 yr.	8″ 24.7 yr.	8″ 26.1 yr.	8″ 28.6 yr.
Fair	8″ 4.9 yr.	8″ 4.9 yr.	8″ 5.0 yr.	8″ 5.1 yr.	8″ 5.1 yr.	8″ 5.2 yr.	8″ 5.5 yr.	8″ 5.0 yr.	8″ 6.1 yr.	8″ 6.4 yr.	8″ 6.9 yr.	8″ 7.1 yr.	8″ 11.0 yr.	8″ 11.6 yr.	8″ 12.5 yr.
Poor	9″ 3.0 yr.	8″ 3.1 yr.	9″ 3.1 yr.	8″ 3.1 yr.	8″ 3.2 yr.	9″ 3.3 yr.	9″ 3.7 yr.	9″ 3.6 yr.	8″ 3.7 yr.	8″ 4.0 yr.	8″ or 9″ 4.1 yr.	9″ 4.5 yr.	8″ 6.8 yr.	8″ 7.1 yr.	8″ 7.7 yr.
Very Poor...	9″ 0.8 yr.	9″ 0.9 yr.	9″ 0.9 yr.	9″ 0.9 yr.	9″ 1.0 yr.	10″ 0.9 yr.	9″ 0.9 yr.	9″ 1.0 yr.	9″ 1.1 yr.	9″ 1.1 yr.	9″ 1.2 yr.	9″ 1.3 yr.	9″ 2.0 yr.	9″ 2.0 yr.	9″ 2.2 yr.

*Estimated commercial traffic intensity distributions for these fifteen samples appear in Table 15.

†See pages 4 and 5.

Table 15

CONVERSION OF COMMERCIAL TRAFFIC INTENSITY TO EQUIVALENT 5,000-POUND WHEEL LOADS FOR PAVEMENT THICKNESSES OF 7, 8, 9, AND 10 INCHES

		Pavement Thickness							
		7″		8″		9″		10″	
Wheel Load (Pounds)	*Estimated Commercial Traffic Intensity Distribution (Percent)*	*Percent Factor*	*Equivalent 5,000-lb. Wheel Loads*	*Percent Factor*	*Equivalent 5,000-lb. Wheel Loads*	*Percent Factor*	*Equivalent 5,000-lb. Wheel Loads*	*Percent Factor*	*Equivalent 5,000-lb. Wheel Loads*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
3,000 ...	30.3%	....	.....	....	.....	0.01%	.00003	0.04%	.00012
4,000 ...	16.4	0.03%	.00005	0.05%	.00008	0.05	.00008	0.09	.00015
5,000 ...	7.9	0.09	.00007	0.13	.00010	0.16	.00013	0.24	.00019
6,000 ...	10.1	0.39	.00039	0.39	.00039	0.40	.00040	0.51	.00052
7,000 ...	10.0	0.95	.00095	0.77	.00077	0.79	.00079	0.97	.00097
8,000 ...	9.6	2.40	.00230	1.50	.00144	1.10	.00106	1.30	.00125
9,000 ...	9.6	5.10	.00489	2.90	.00278	2.30	.00222	1.90	.00181
10,000 ...	4.6	9.20	.00423	5.20	.00240	3.80	.00175	3.00	.00138
11,000 ...	1.2	15.20	.00183	8.20	.00098	5.90	.00071	4.60	.00055
12,000 ...	0.2	25.60	.00051	13.60	.00027	9.30	.00019	5.80	.00012
13,000 ...	0.07	48.60	.00034	23.60	.00017	17.20	.00012	9.70	.00007
15,000 ...	0.03	138.00	.00041	63.30	.00019	39.80	.00012	23.10	.00007
Total ..	100.00%	....	.01597	....	.00957	....	.00760	....	.00720

The example demonstrates that choosing the proper depth of slab is not difficult if traffic intensity can be estimated and percent factors for subgrades and depths of pavement are known.

Advantages from improving subgrade support are indicated by Table 14—improving a subgrade from a poor to a very good classification will increase the pavement life five to six times. This would justify improving subgrades for concrete pavements until they reach either the good or very good classification. The charts also indicate that 8″ pavement depths are sufficient if the subgrade is good or very good.

Concrete pavements can be made of lesser depth than is now the custom in Pennsylvania and yet have a longer life than existing pavements if proper procedure is followed to improve the bearing capacity and lower the water table of the subgrade. Each section of highway requires individual treatment in design and construction.

APPENDICES

Appendix A

GEOMETRIC DESIGN STANDARDS

The geometric design standards which follow are intended for use with pavement thickness and subgrade design data in examination of comparative costs of highway construction and pavement replacement under various loadings.

In practical design, general rules cannot be applied in every case, since each highway plan presents its own problems. Design speeds in the standards listed here are included to provide for examination of relative costs for various design speeds in all traffic ranges. Some of the lower design speeds would, of course, be ignored in practical design for heavy traffic classifications.

Wider pavements and longer sight distances provide more satisfactory operating conditions. Recent studies show that increasing the lane width reduces stresses in rigid pavements by precluding an excess of edge loading and prevents undesirable "tracking" on flexible pavements. Further, important gains in pavement life accompany increases in lane width.

Table A-1
GEOMETRIC DESIGN STANDARDS

Class of Highway	*Daily Average of Yearly Traffic*	*Design Speed (Miles per Hour)*	*Recommended Sight Distances*		*Recommended Design Width (feet)*
			Passing (feet)	*Nonpassing (feet)*	
(1)	(2)	(3)	(4)	(5)	(6)
1	10,000 to 20,000	40	....	275	48*
		50	....	350	48*
		60	....	475	48*
		70	....	600	48*
2	5,000 to 10,000	40	....	275	36
		50	1100	350	36
		60	1500	475	36
		70	2000	600	36
3†	1,500 to 5,000	40	1100	275	22
		50	1600	350	22–24‡
		60	2300	475	22–24‡
		70	3200	600	24
4†	800 to 1,500	40	1100	275	22
		50	1600	350	22
		60	2300	475	24
		70	3200	600	24
5†	400 to 800	40	1100	275	20
		50	1600	350	22
		60	2300	475	22
		70	3200	600	24
6	200 to 400	40	1100	275	20
		50	1600	350	20
7	Up to 200	40	1100	275	20
		50	1600	350	20

*Consisting of four twelve-foot lanes, divided.
†On highways of Classes 3, 4, and 5, where grade exceeds 4 percent, truck lane is recommended if length of grade is over one-fourth mile.
‡Varies with volume of truck traffic.

Appendix B

HISTORY OF ANALYSIS AND DESIGN OF PORTLAND CEMENT CONCRETE PAVEMENTS

The analysis and design of a rigid pavement has often been likened to the analysis and design of a rigid plate resting on an elastic foundation. The plate pattern of analysis has been supplemented by laboratory experiments in which steel plates are used on truly elastic foundations, and by field tests on full-size pavement slabs placed on carefully prepared subgrades.

Work on the theory of plates may be traced to Germain (B-7), who presented findings in 1821. A better-known publication is that made by Navier (B-36) about 1823. A few years later, Poisson (B-20) and Kirchhoff (B-15) presented papers which appeared to be contradictory. It remained for Thompson and Tait (B-35), and then Boussinesq (B-3), to show that Poisson's theory applied to thick plates, and Kirchhoff's to thin plates. In 1884, Hertz (B-11) presented his work on floating loaded plates, and Ritz (B-24) followed with his method for the solution of certain elasticity problems. Happel (B-10) and Murphy (B-16), using the results of the work of Hertz and Ritz, studied the problem of load stresses in rectangular plates resting on elastic foundations. In relatively recent years, papers dealing with plates resting on semi-infinite elastic foundations have been presented by Hogg (B-12), Holl (B-13), Pickett (B-18), and Pickett and Ray (B-19).

A direct approach to the problem of required pavement thicknesses for support of definite loads was made by Westergaard (B-40) in 1925. In 1927, Westergaard presented a paper on the theory of stresses in pavement slabs caused by restraint of expansion or contraction due to temperature or moisture effects (B-38). In 1933, in conjunction with tests being conducted by the Bureau of Public Roads, Westergaard presented a third paper (B-39) which contained a modification of his earlier equation for maximum load stress in a slab when wheel loads are applied at a considerable distance from the edge of the pavement. These three contributions of Westergaard, together with the results of the Arlington road tests, have led to direct determinations of the major stresses in concrete pavements subjected to particular loading, subgrade, and temperature conditions.

The results of the Arlington road tests, initiated and planned to check Westergaard's work, were presented by Teller and Sutherland (B-30 to 34). Kelley (B-14) correlated the results of the Arlington tests and the theoretical work of Westergaard. Kelley also reviewed the work of Older and the results of the Bates road tests, which had been conducted in 1922-23 in Illinois, and presented empirical equations for use in evaluating the maximum load stresses in concrete pavement slabs. Kelley's data indicate that the maximum restrained warping stresses in slabs over 30 feet long are very great and will appreciably affect the total stress at the free longitudinal edge of the slab. Bradbury (B-4) demonstrated that, for the same numbers of critical stress repetitions, a slab may crack transversely some distance from the corner because of longitudinal-edge stresses before it will crack across the corner because of corner stresses. Bradbury, like Kelley, recommended empirical modifications of Westergaard's equations, and supplied charts of coefficients to facilitate solution of load stress or restrained warping stress equations.

Pickett presented an empirical equation for critical stresses at a section near the corner of a slab due to a load applied at the corner. Curling stresses (stresses due to restrained warping) at such a section are very small in comparison with load stresses, and there appears to be merit in Pickett's development of the empirical equation of curves having the general shape of the Westergaard curve but passing through the plotted points of the Arlington road tests.

Spangler and Lightburn (B-27), who conducted studies and tests at Iowa State University, found that Westergaard's assumption of uniform subgrade support beneath all portions of the slab was not completely correct, but that Westergaard's equations using a uniform k value provided stress values which agreed closely with those computed from measured strains. Spangler (B-25) also pointed out that Westergaard's assumption of a uniform unit stress across the length of a section normal to the bisector of the corner angle (for the corner loading condition) was not precise. In a later paper (B-26), Spangler (whose papers were concerned chiefly with corner loading) stated that Kelley's equation for corner stresses more closely matched the results of the Iowa tests than did Westergaard's equation.

The foregoing papers are directly concerned with slab thickness. Design of the slab thickness cannot be isolated from consideration of joint design. Among papers on this subject are those of Van Breeman (B-37) and Anderson (B-1). The research report of Jensen, West, Loewer, and de Neufville (B-5), of Lehigh University, contains a comprehensive review of the literature in this field.

The final report on Road Test One-MD (B-6)—the Maryland road test—provides information concerning divergences between current theory and practice. In Road Test One-MD, the relative destruction of the Portland cement concrete pavement (of a 9-7-9 inch cross section) under various axle loads was observed. Strain measurements were made during the day and night with vehicles located at different parts of the slabs. Load stresses and restrained warping stresses due to temperature were calculated from measured strains. Since the fatigue resistance of concrete in bending has usually been found by conducting bending tests on concrete beams of a limited width, the high stresses calculated from the measured strains in the Maryland road test indicate need for further research to determine the ability of concrete slabs to resist localized bending stresses above 55 percent of the modulus of rupture of the concrete.

Summarizations of certain especially significant studies in highway design and analysis, with pertinent comments, are presented below.

Westergaard

In his initial analysis, in 1926, Westergaard assumed that a pavement slab would act as a homogeneous, isotropic, elastic solid in equilibrium and that reactions of the subgrade were vertical and proportional to deflections of the slab.

Westergaard considered three possible positions of load: (a) applied as close as a wheel could be placed to the corner, (b) applied at the longitudinal edge at an appreciable distance from the corner, and (c) applied at a considerable distance from any edge (interior loading). For longitudinal-edge and interior loading, the maximum load stress will occur beneath the applied load, but for corner loading the maximum load stress will occur a few feet away from it.

For corner loading, the Goldbeck equation (B-17) shows the stress along the bisector of the corner angle as: $\sigma_c = \frac{3P}{d^2}$. This equation was derived from consideration of a load applied exactly at the corner with the subgrade beneath the slab corner missing. Westergaard reasoned that the wheel load, P, should be thought of as being distributed over its contact area with the pavement so that, with the edge of the wheel just on the edge of the pavement, the center of gravity of the load would be at a distance of a_1 from the corner itself. Thus, by use of a coordinate system with the origin at the corner and the X axis bisecting the corner angle, and by use of Ritz's method of successive approximation to solve the fourth-order differential equation governing the flexure of the slab, Westergaard obtained an equation for the maximum stress parallel to the bisector of the corner angle and at a distance $2\sqrt{a_1 l}$ from the corner. This equation is:

$$\sigma_c = \frac{3P}{d^2}\left[1 - \left(\frac{a_1}{l}\right)^{0.6}\right] \qquad \text{(V)}$$

$$\text{where } l = \sqrt[4]{\frac{Ed^3}{12(1-\mu^2)k}} \text{ (radius of relative stiffness)}$$

To obtain the foregoing equation, Westergaard further assumed that the critical section for maximum stress was one perpendicular to the bisector of the corner angle, that the stress was uniform across this section, and that the subgrade supported the slab at all points. A number of years later, Kelley and Spangler (B-27) offered different opinions concerning the shape of the critical section when a load was applied near the corner. The Arlington road tests, however, confirmed Westergaard's location of the point of intersection of this critical section and the bisector of the corner angle as being a distance of $2\sqrt{a_1 l}$ from the corner.

Westergaard's equation for the maximum load stress for a load applied in the interior of the slab was not so easily derived as was his equation for the critical stress near the corner for a load applied close to the corner. His equation is:

$$\sigma_{ix} = 0.3162\frac{P}{d^2}\left[\log_{10}d^3 - 4\log_{10}\left(\sqrt{1.6a^2 + d^2} - 0.675d\right) - \log_{10}k + 6.478\right] \qquad \text{(VI)}$$

Conditions:

$a < 1.724\,d$
$\mu = 0.15$
$E = 3(10)^6$ psi

In 1933, Westergaard presented a revised equation for the maximum load stress in the interior of the slab:

$$\sigma_{ix} = 0.275(1+\mu)\frac{P}{d^2}\left[\log_{10}\left(\frac{Ed^3}{kb^4}\right) - 54.54\left(\frac{l}{L}\right)^2 Z\right] \qquad \text{(VII)}$$

This supplementary equation was based on the theory that, for a continuous body, the reactions of the subgrade might be expected to be more closely concentrated around the load than would the deflections. The latter equation generally replaced the former for determination of maximum load stress in the interior of the slab. The principal difficulty encountered in applying Equation VII is the determination of the values which should be substituted for L and Z; L is the maximum value of the radius of the circular area within which a redistribution of subgrade reactions is made, and Z is a type of maximum deflection reduction ratio. Westergaard concluded that the value of Z might range from 0 to 0.39 but could be determined only by experiments on particular road sites, since it depended on slab thickness, subgrade bearing value, and magnitude of applied load. Since experimental values for sites might vary appreciably and since obtaining this data was impractical, Bradbury (B-4) later suggested that a mean value of 0.20 be used for Z. Bradbury also used a value of 0.05 $\left(\frac{l}{L}\right)$ in his sample problems. However, since the values of L and Z will affect appreciably the load-stress magnitude obtained, a method is needed for determining L and Z.

To obtain the equation for maximum load stress along the edge of a slab at an appreciable distance from the corner, Westergaard again used a more involved analysis than that needed to obtain the equation for corner loading. (The stress perpendicular to the edge must, of course, approach zero at the edge, so it is not of significance.) For the load stress parallel to the edge Westergaard obtained the equation:

$$\sigma_{ex} = 0.572 \frac{P}{d^2} \left[\log_{10}(d)^3 - 4 \log_{10} \left(\sqrt{1.6 a^2 + d^2} - 0.675d \right) - \log_{10}k + 5.767 \right] \qquad \text{(VIII)}$$

It is interesting to note that results of the Arlington road test, when loads were applied during the day, agreed closely with those obtained by use of the above equation.

The following were not considered by Westergaard in his 1926 paper on stresses in slabs:

1. Tendency of the slab to change volume owing to variations of temperature and other causes
2. Use of a slab with nonuniform thickness of cross section
3. Localized hard or soft spots in the subgrade
4. Effect of horizontal components of the reactions of the subgrade.

In 1927, Westergaard presented a paper on the analysis of stresses in concrete pavements due to restraint of free warping caused by the slab weight. Assuming that a uniform temperature gradient existed throughout the thickness of the slab, Westergaard derived equations for the stress in either direction in the interior of a slab of infinite length and infinite width and for the stress in a longitudinal direction at the edge of a slab with infinite length and finite width. Bradbury, however, expressed Westergaard's equations for both of these cases in simpler form, using coefficients for which he had prepared tables. In this form they appear as

$$S_{ix} = \frac{Ee\,\Delta t}{2}\left(\frac{C_x + \mu C_y}{1 - \mu^2}\right) \qquad \text{(IX)}$$

$$S_{ex} = \frac{C_x Ee \Delta t}{2} \qquad \text{(II)}$$

where C_x and C_y are coefficients involving the length and width of the slab. Kelley (B-23) also treated warping (slab curling) stresses[1] and arrived at equations which are slightly different from Bradbury's but which give identical answers.

In 1937, an article by Westergaard (B-41) explained that his equations for σ_{ix} and σ_{ex} were obtained by formal integration of the fourth-order partial differential equation governing flexure of the slab and pointed out that the equation for σ_c was obtained in a much simpler manner. He then considered the case of corner loading when 100 percent subgrade support is not attainable at the corner, and reported that the maximum unit load stress at the critical section near the corner was increased by 11 percent when there was no support under the corner or for a distance, a, along each side.

Teller and Sutherland

A series of tests was conducted by the Bureau of Public Roads at Arlington, Virginia, in 1930 (B-30) to study four principal subjects:

1. The effects of loads placed in various ways on pavement slabs of uniform thickness.
2. The 'balance of design' or relative economy of typical pavement slab cross-sections.

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[1] The term "warping stresses" is used synonymously with "restrained warping stresses." There are no true "warping stresses."

3. The behavior under load and comparative structural effectiveness of typical longitudinal and transverse joint designs.
4. The effects of temperature conditions and moisture conditions on the size, shape, and load-carrying ability of pavement slabs.

The study of the effects of loads placed in various ways on slabs of uniform thickness was intended primarily as an experimental verification of the only rational theory of pavement slab stresses thus far advanced, i.e., the Westergaard analysis.

Ten slabs, each 40 feet long and 20 feet wide, were constructed. Each slab was divided by a longitudinal joint, and none of the slabs contained distributed reinforcement. The subgrade was carefully prepared, and was made uniform beneath the ten slabs. Loads were applied through a device equipped with a removable bearing plate, and this was the only load on the slab at the time of each particular test. The slabs were shaded to prevent warping due to temperature differentials in the slab when such warping was not being considered. With the foregoing preparations, all three positions of loading which Westergaard had investigated were used: corner, free longitudinal edge, and interior. To conform to Westergaard's method of applying the loads, a circular bearing plate was used for corner and interior loadings and a semicircular one for edge loadings.

The experiments showed that equations used to calculate maximum load stress at night for a load along the edge of a pavement could not be used satisfactorily to obtain maximum load stress at the same point with the identical load during the day. This was due to the differential in temperature between the top and bottom of the pavement slab which tends to curl or warp the edge of the slab upward at night and downward during the day. It was found that the maximum load stress when the load was applied on a free longitudinal edge warped up or down was from 5 percent to 20 percent higher or lower (night or day, respectively) than the maximum load stress if the slab were flat and no warping or curling existed. An important effect was pointed out: The weight of the slab tends to restrain free warping of the slab, so that during the night the edges of the slab are prevented from curling upward as far as they might if no restraint existed and so that a tensile stress is developed in the top of the slab with an accompanying compressive stress in the bottom. During the daytime, the slab edges tend to curl downward so the slab weight will produce tensile stresses in the bottom of the slab and compressive stresses in the top of the slab. This restrained warping stress is of appreciable magnitude; in fact, it is often as great or greater than the maximum load stress in a pavement slab. Since the temperature differential in the slab during the day is from two to three times that at night (see Table A-2), the tensile stress due to restrained warping is much greater during the daytime. A differential in moisture content between the top surface of the slab and the subgrade beneath the slab will also tend to produce warping, but the Arlington tests indicated that the restrained warping stresses due to moisture differential were very small and tended to reduce stresses caused by restrained warping due to temperature differentials.

Teller and Sutherland presented the report on the Arlington road tests in five parts; the conclusions of the second part (on temperature and moisture effects upon stresses) were (B-31):

1. The average pavement temperature undergoes an annual change of about 80°F.
2. The maximum temperature differentials observed at the edges of the test sections were:
   a. For a 6-inch uniform thickness section, 23°F.
   b. For a 9-inch uniform-thickness section, 33°F.
   c. For a 9-6-9 thickened-edge section, 33°F.

Table A-2

MAXIMUM OBSERVED TEMPERATURE DIFFERENTIALS: ARLINGTON ROAD TEST

I. Summary of Maximum Positive Temperature Differentials: Twenty-seven Days Between April 3 and June 4, 1934—

Type of Slab and Point of Observation	*Temperature Differential (Degrees Fahrenheit)*		
	Maximum	*Minimum*	*Average*
(1)	(2)	(3)	(4)
Uniform 6-inch at edge	+23	+14	+19
Uniform 9-inch at edge	+33	+20	+27
9-6-9 inch			
at edge	+33	+18	+27
18 inches from edge	+31	+17	+25
36 inches from edge	+28	+15	+22

II. Summary of Maximum Temperature Differentials: Seventeen Days During 1931, 1932 and 1933—

Type of Slab and Point of Observation	*Temperature Differential (Degrees Fahrenheit)*		
	Maximum	*Minimum*	*Average*
(1)	(2)	(3)	(4)
Uniform 6-inch			
April to August (inclusive)			
Day	+24.3	+18.7	+21.2
Night	— 6.5	— 4.5	— 5.8
September to February (inclusive)			
Day	+15.6	+ 8.2	+11.8
Night	— 6.7	— 1.3	— 4.1
Uniform 9-inch			
April to August (inclusive)			
Day	+31.0	+22.3	+26.9
Night	— 9.2	— 5.7	— 7.5

Source: *Public Roads*, Vol. 20, No. 5, (July 1939), p. 97.

These maxima occur during the hot afternoons of early summer when the upper surface of the pavement is heated by the intense sunlight and the lower surface is kept cool by a subgrade that is still at a relatively low temperature.

3. In the thickened-edge design the temperature differential in the interior of the slab averaged about 4°F. less than that at the thickened edge during the most critical part of the year.

4. There is a cyclic variation in slab length that is entirely dissociated from temperature changes. The annual variation in the length of the test sections from causes other than temperature changes is approximately equivalent to that caused by a temperature change of 30°F., and the maximum length occurs during the late winter when the ground moisture content is greatest. Conversely, the slab is shortest during the late summer when the ground moisture and, so far as could be determined, the concrete moisture are a minimum.

5. The thermal coefficient of expansion of the concrete as determined in the laboratory is 0.0000048 per degree F. This value agrees almost exactly with that determined by measurement of actual temperature expansion in the test sections, indicating: First, that the movement of a pavement slab from thermal expansion can be predicted accurately from laboratory determinations of the thermal coefficient; and second, that in slabs of moderate length the effect of subgrade restraint on slab expansion is so small as to be negligible.

6. The resistance developed in the subgrade to horizontal slab movement is not merely a matter of sliding friction in the commonly accepted sense of the word. It appears to consist of two elements, one an elastic deformation of the soil horizontally that is present for all displacements of the slab, and the other a frictional resistance that develops only after a certain amount of elastic deformation has occurred. The first element appears to be independent of, while the second varies directly with, the slab weight or thickness. Although only one subgrade material was involved in these tests, it seems probable that the relative importance of the two elements may vary considerably with different types of soils.

7. In pavement slabs of moderate length the tensile stresses resulting from contraction will not be large for subgrade soils of the type used in these tests. The thicker the pavement the lower will be the unit stress from this cause, other conditions being the same.

8. The changes in shape of a pavement slab resulting from restrained temperature warping do not cause large changes in the critical stresses from applied loads. In this investigation, the maximum observed condition of upward warping from temperature was found to increase the critical stress resulting from load by about 5 percent for a corner loading and about 20 percent for an edge loading, as compared with the stresses produced by the given load with the slab in the flat or unwarped condition. Maximum downward warping was found to effect a negligible reduction in the load stress at the edge and a reduction of about 20 percent at the corner.

9. For pavement slabs of the size used in this investigation or larger, certain of the stresses arising from restrained temperature warping are equal in importance to those produced by the heaviest of legal wheel loads. The longitudinal tensile stress in the bottom of the pavement, caused by restrained temperature warping, frequently amounts to as much as 350 pounds per square inch at certain periods of the year and the corresponding stress in the transverse direction is approximately 125 pounds per square inch. These stresses are additive to those produced by wheel loads.

10. In long or even moderately long pavement slabs, when conditions are such as to produce large temperature differentials, thickening the edge of the slab may actually decrease the load-carrying capacity of this part of the pavement. In very short pavement slabs, thickening the edge of the slab may be expected to increase definitely its load-carrying capacity.

11. Since the critical stresses resulting from restrained warping are opposite in sign to those caused by applied loads in the corner region of a pavement, thickening the edge of the slab may be expected to increase the load-carrying capacity of the slab corner.

12. Because of the facts stated in conclusions 10 and 11, it is evident that thickening the edge of a long pavement slab will not tend to reduce transverse cracking but will tend to reduce corner cracking.

13. The annual cyclic variation in moisture conditions within the concrete produces a warping of the slab surface similar to that caused by temperature. The edges of the slab reach their maximum position of upward warping from this cause during the summer and the maximum position of downward warping during the winter, the extent of the upward movement apparently exceeding that of the downward movement considerably.

14. While sufficient information is not available to permit an estimate to be made of the magnitude of the stresses arising from restrained moisture warping, it appears that at the time of year when the stresses from restrained temperature warping are a maximum (the summer months) any stresses caused by restrained moisture warping will be of opposite sign and will thus tend to reduce rather than to increase the state of stress created by restrained temperature warping.

Part III of the report by Teller and Sutherland—a resumé of a study of concrete pavement cross sections—contained the following statement (B-32):

The results of this study are surprising as to the stresses that will exist in concrete pavements with certain combinations of load, temperature conditions, and slab thickness. However, the conclusions are thought to be sufficiently well established for application in current design.

If loads alone are considered, the maximum of economy in the use of material is obtained with a thickened-edge cross section.

While increased edge thickness results in a reduction of the edge stresses from applied load, it also causes an increase in the edge stresses under certain conditions of restrained warping.

Since a balanced cross section should in all cases be designed on the basis of combined load and warping stresses, it is obvious that economy demands that the stresses resulting from warping be limited to low values. The most practical way of doing this is by constructing short pavement slabs.

In short slabs the cross section may be designed on the basis of load alone.

The foregoing statement points out directly that warping stresses should not be ignored unless the slab is short. It also emphasizes that increased edge thickness causes an increase in restrained warping stresses.

In this part of the report the effect of multiple-wheel loading was also discussed. Axles with wheels 21-45-21 inches apart and also with wheels 28-28-28 inches apart were considered, and the maximum load stress in the slab beneath a wheel was found to be about 1.3 times as much as it would have been had the wheel been on the slab by itself. Cross-sections used in this part of the test were the 6-inch uniform-thickness slab and the 9-6-9-inch slab.

The fourth part of the Teller and Sutherland report was concerned with transverse and longitudinal joint design. Until 1917, load transfer was not considered a major factor in joint design; the dominant considerations prior to that time were protection of joint edges and expansion. However, the practice of using doweled joints prevailed after satisfactory installation of four ¾-inch dowels across ⅜-inch joints in a 20-foot width of pavement near Newport News, Virginia, in 1917.

The spacing of transverse joints remains an area of research. In 1934, the Bureau of Public Roads stipulated that in Federal-aid road construction, expansion joints should be placed at intervals of not more than 100 feet and transverse joints (in plain concrete slabs) should be placed at intervals not exceeding 30 feet. The required width of expansion joints was from ¾ inch to 1 inch, and provision for load transfer was required in all transverse joint installations.

The summary of Part IV of the Teller and Sutherland report (B-33) is as follows:

Joints are needed in concrete pavements for the one purpose of reducing as much as possible the stresses resulting from causes other than applied loads in order that the natural stress resistance of the pavement may be conserved to the greatest possible extent for carrying the loads of traffic.

A joint is potentially a point of structural weakness and may limit the load-carrying capacity of the entire pavement.

Joints are classified by function as:

1. Those designed to provide space in which unrestrained expansion can occur.
2. Those designed for the relief or control of the direct tensile stresses caused by restrained contraction.
3. Those designed to permit warping to occur, thus reducing restraint and controlling the magnitude of the bending stresses developed by restrained warping.

Expansion joints should be provided at no greater intervals than about 100 feet in order to keep the joint openings from becoming excessive.

The spacing of contraction joints will be determined by the permissible unit stress in the concrete. If this is restricted to a low value, which is desirable, contraction joints should be provided at intervals of about 30 feet.

It is indicated that joints to control warping should be spaced at intervals of about 10 feet.

> A free edge is a structural weak spot in a slab of uniform thickness, and it is necessary to strengthen the joint edges by thickening the slab at this point or by the introduction of some mechanism for transferring part of the applied load across the joint to the adjacent slab.
>
> The doweled transverse joints investigated were quite effective in relieving stresses caused by expansion, contraction, and warping, but they were not particularly effective in controlling load stresses near the joint edge.
>
> The dowel-plate joint tested had merit as a means for load transfer, though it offered more resistance to expansion and contraction than is desirable.
>
> Aggregate interlock as it occurs in weakened-plane joints cannot be depended upon to control load stresses. Even when joints of this type are held closely by bonded steel bars there is wide variation in the critical stress value caused by a given load; therefore, it appears necessary to provide independent means for load transfer in plane-of-weakness joints.
>
> Tongue-and-groove joints held together by bonded steel bars were found to be the most efficient structurally of any of the joints studied. However, modifications of the designs might improve their action.

Improper functioning of joints has been a serious problem. It is difficult to make joints waterproof, and "growing" of pavements, resulting from collection of extraneous material in contraction joints, with subsequent "freezing" of doweled joints, has also been quite common. In recent years, projects have been initiated in several states for observation and study of slab movements and expansion joints. Early reports from these experiments are not conclusive. The Arlington tests revealed that weakened-plane contraction joints were ineffective for load transfer and that dowels, to be effective, must be large and spaced relatively closely, with small joint openings.

The last part of the Arlington report was not published until 1943. Prior to that time, Kelley presented a paper containing the major results of this last part and coordinating these with the research of others.

Several supplementary tests were made on the pavement slabs at Arlington, among them a comparison of load stresses obtained with circular loaded areas and with elliptical bearing areas. The result of this examination was as follows (B-34):

> In the past, for purposes of stress computation for highway loadings, it has often been assumed that the actual tire contact area should be represented by the circle of equivalent area. It is interesting to note that for areas such as are found in highway service, the data indicate the error resulting from this assumption would be small, probably 5% or less. For the larger areas the percentage would be slightly more.

A general conclusion of the report by Teller and Sutherland on the Arlington road tests was that, within the limits of the investigation and as long as the basic conditions assumed for the analysis are approximated, the Westergaard theory accurately described the action of the pavement.

Kelley

In 1939, Kelley presented a resumé of information pertaining to analysis and design of Portland cement concrete pavement. His paper stressed the results of the Arlington road tests.

Kelley referred to investigations by Teller in ascertaining if adjacent wheels should be considered in determining maximum load stress beneath a particular wheel. Teller had found that, for the four- or six-wheel truck, the maximum load stress developed in a concrete pavement beneath a particular wheel was a function of that wheel load alone, provided the wheels were spaced at least three feet apart. Although an *axle* spacing closer than this in a truck is impossible, several *wheels* can be placed on an axle so that they are closer together than three feet. (Dual wheels are counted as one wheel.) For wheels spaced at less than three feet, the load stress under a particular wheel could be 1.3 times that under a wheel with spacings greater than three feet. Westergaard's original paper contained a problem in which the four wheels of a two-axle truck were considered; he also found that the negative effect of some wheels offset the small positive effect of others, so that the total load stress beneath a wheel might be found by considering that wheel alone.

Questions of quantification and use of an impact factor have not been answered completely. Early experiments on this subject indicated that the impact factor for heavy loads was less than that for lighter loads, and factors of the order of 1.2 were found applicable for trucks which had wheel loads of about 10,000 pounds on dual balloon tires if the truck passed over an obstruction on the road (B-29). With the improvement of spring-suspension systems in trucks in the last two decades, the impact factors of the 1920's are not considered applicable. The report of Road Test One-MD showed that a negative impact factor probably should be used if the truck is to travel at a speed of 30 to 40 mph, since measured strains in pavement slabs reduced when speeds were increased.

Kelley recommended that the following equations be used to determine load stresses:

$$\sigma_c = \frac{3P}{d^2}\left[1 - \left(\frac{a_1}{l}\right)^{1.2}\right] \qquad \text{(X)}$$

$$\sigma_{ex_{(day)}} = 0.57185\,\frac{P}{d^2}\left[4\log_{10}\left(\frac{l}{b}\right) + 0.3593\right] \qquad \text{(I)}$$

$$\sigma_{ex_{(night)}} = 0.57185\,\frac{P}{d^2}\left[4\log_{10}\left(\frac{l}{b}\right) + \log_{10} b\right] \qquad \text{(III)}$$

$$\sigma_{ix} = 0.31625\,\frac{P}{d^2}\left[4\log_{10}\left(\frac{l}{b}\right) + 1.0693\right] \qquad \text{(XI)}$$

Equation X and Equation III were obtained from the Arlington test results. Equation I is Westergaard's equation. Kelley stated that Equation XI (a variation of Westergaard's original equation for interior load stress) would give conservative yet not uneconomical values and observed that the values of *Z* and *L* which should be used in Westergaard's revised equation could be found only by tests on large slabs resting on the same type of soil and having conditions of load and slab thickness identical to those of the pavement to be constructed. Kelley acknowledged that Bradbury had determined that all four of the above equations, as well as Westergaard's revised equation for σ_{ix}, were of the form: $\sigma = (C)\,P/d^2$. For the interior and free longitudinal-edge stress, the coefficient C is fixed by the ratio of l/b, and for the corner loading by the ratio of a_1/l. Bradbury prepared tables of these coefficients (C_e, C_i and C_c), but it must be noted that for a particular pavement design in a specific location it is difficult to predetermine the precise values to be assigned to the variables upon which these coefficients depend. Kelley suggested that the modulus of elasticity of concrete be assumed to be about $5(10)^6$ psi and that the subgrade modulus, *k*, be assumed as 100 psi/in when test data were lacking.

Kelley stated that restrained warping stresses or curling stresses (due to the temperature differential between the top and bottom of the slab) were affected greatly by even a moderate change in air temperature. Warping stresses in the longitudinal direction of the slab may be very great and, when they are combined with maximum load stress in the same direction, critical stresses which tend to cause transverse cracks may be reached. The greatest warping stresses in the fall and winter are only about two-thirds of the greatest occurring in the spring and summer, and these maximum values of warping stress exist only a few hours of each day. Therefore, the frequency of critical stresses resulting from combinations of maximum load stress and maximum warping stress is smaller than might initially be anticipated.

Kelley pointed out that the warping stress in the transverse direction for a 10-foot slab width was only about one-half that for a 20-foot slab width. From this, the advantage of using longitudinal joints is apparent.

Kelley noted that both theory and experiment indicate that the warping stress is zero at the corner and increases as the distance from the corner along the corner bisector increases. The important warping stress is that which occurs at the same place and at the same time, and which has the same sign, as the maximum load stress. At night, when the slab is warped upward, the load stress and warping stress at the critical section near the corner are of the same sign; therefore, the warping stress tends to increase the combined stress. However, the stress is not great, since at night the temperature differential and the resultant warping stress are small. For a temperature differential at night of one degree F. per inch of slab thickness (which was suggested by both the Arlington and Road Test One-MD reports), a warping stress of about 40 psi, only, would be introduced.

In contrast to this, the warping stresses along the free longitudinal edge and in the interior of the slab are not small. These stresses increase directly with the slab thickness and are affected by the length of the slab in the direction of stress. For slabs longer than 25 or 30 feet, the warping stress in the longitudinal direction remains almost constant at a high value. Although the warping stress continues to increase as the slab depth increases when length is greater than 25 or 30 feet, the depth of the slab has a more pronounced effect on the warping stress for shorter slabs. It can be seen from Table A-3 that the length of the slab must be reduced to 20 feet or less if the warping stress at the free longitudinal edge is to be kept to a magnitude well below 50 percent of the modulus of rupture of the concrete. The following table from Kelley's paper shows combined load and warping stresses which may exist in the interior or along the longitudinal edge of a slab 8 inches deep and 10 feet wide when a movable wheel load of 11,800 pounds is applied:

Slab Length (feet)	*Combined Longitudinal-Edge Stress (psi)*		*Combined Interior Stress (psi)*	
	$k = 100$	$k = 300$	$k = 100$	$k = 300$
30	670	610	570	550
15	530	570	430	510
10	400	430	300	370

From the table it is evident that unless very short slabs are used, high combined stresses are developed. Examination of Kelley's paper indicates that, although load stress decreases as slab thickness increases, the *combined* longitudinal-edge stress or *combined* interior stress does not always decrease as slab thickness increases. With a slab length of 20 feet, these combined stresses increase as slab depth is increased beyond 9 inches.

Kelley agreed with the conclusion of Teller and Sutherland that warping stresses caused by moisture differentials are small and usually opposite to those caused by temperature differentials and therefore may be neglected without introducing appreciable error.

Kelley advocated the use of a uniform cross section. However, recognizing that for heavy design loads and slab lengths shorter than 15 feet the thickened-edge cross sections were to be preferred, he suggested the relationship of 1.67 between the interior and edge thicknesses. This relationship of 1.67

Table A-3

MAXIMUM LONGITUDINAL WARPING STRESSES OBSERVED IN SLABS TEN AND TWENTY FEET IN LENGTH: ARLINGTON ROAD TEST

Month and Day (1934)	*Maximum Air Tempera-ture (°F.)*	*Maximum Longitudinal Warping Stress (psi)*				*Percent Reduction in Stress Accompanying Decreased Slab Length*	
		In Interior		*At Edge*			
		20-foot Slab	*10-foot Slab*	*20-foot Slab*	*10-foot Slab*	*Interior*	*Edge*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
April 26	68	307	132	...	..	57%	..
May 1	74	376	142	...	..	62	..
May 2	71	...	...	121	68	..	44%*
May 13	83	287	81	...	..	72	..
May 14	90	...	...	278	21	..	92
May 28	83	429	151	...	..	65	..
June 1	90	...	...	354	46	..	87
June 11	94	...	...	285	38	..	87
June 14	88	...	...	313	20	..	94
June 15	94	...	...	252	19	..	92
June 21	101	451	132	...	..	71	..
June 22	96	414	130	...	..	69	..
June 25	97	...	...	283	51	..	82
Average						66	89

*Not included in average.

appeared for slab lengths less than 15 feet in the Arlington road tests. Kelley did not recommend definite joint spacings but stated that if 10-foot spacings were used the corner stresses and the edge stresses would be about equal, and under these conditions he noted no need for load transfer devices.

The fatigue limit of concrete was given extensive consideration, and Kelley's conclusion was that a working stress of somewhat more than 50 percent of the ultimate strength of the concrete was conservative if actual combined stresses of load and warping were considered. In a pavement slab subjected to warping and load stresses, the maximum combined stress—along the free longitudinal edge—will be localized (existing only beneath the moving load). Conventional fatigue tests on concrete do not apply to this type of localized bending stress. There is doubt that curves plotted from conventional fatigue tests should be used to estimate the numbers of repetitions of localized stresses which will cause failure. This same point has been emphasized in the final report on Road Test One-MD. According to unofficial estimates, a working stress up to 70 percent of the modulus of rupture of concrete might be repeated an exceedingly large number of times before failure occurred. However, there are no test results to verify this, and it is not an official estimate of the Highway Research Board.

The conclusions contained in Kelley's paper—a comprehensive and critical study of the theoretical and experimental data available in 1939, before many changes in truck design—are as follows:

1. The critical load stresses developed in a concrete pavement are primarily dependent on single wheel loads and *not* on axle loads, axle spacing *or the gross weight of vehicle.*

2. Impact forces considerably in excess of static wheel loads should be used in the design of pavements. The impact factor (ratio of total impact reaction to static wheel load) is less for balloon tires than for high-pressure tires and decreases as the wheel load increases.

3. The stresses in a concrete pavement are approximately the same for an 8,000-pound wheel load on dual high-pressure tires and for a 9,000-pound wheel load on dual balloon tires.

4. The stress analyses of Westergaard, with the modifications suggested by the Arlington tests, are suitable for use in the design of concrete pavement slabs and form the only adequate basis for such design.

5. Since the physical characteristics of the subgrade and of the concrete can never be foretold with certainty it is desirable to be conservative in the selection of values representing these various characteristics for use in design.

6. Warping stresses due to differentials of temperature within the slab may be of the same order of magnitude as the stresses due to heavy wheel loads and therefore require consideration in pavement design.

7. Reasonable assurance of the absence of transverse cracking in concrete pavements can be obtained only by use of short slabs having lengths not greater than 10 to 15 feet.

8. Transverse cracks in thickened-edge pavements without reinforcement create a weakened condition in the interior of the slab which may be serious. The introduction of properly designed steel reinforcement in long slabs will not completely eliminate transverse cracking but it will reduce or eliminate the detrimental effect of the cracks which may develop.

9. The edges of transverse joints in thickened-edge slabs require strengthening because the central portion of the joint has the same thickness as the interior of the slab but is subjected to the higher stresses that are associated with edge loading.

10. When the pavement is designed for the combined stresses due to load and temperature it is safe practice to use an allowable unit stress in excess of 50 per cent of the 28-day flexural strength of the concrete.

11. When the pavement is designed for maximum legal wheel loads and in such manner that the combined stresses due to load and temperature are limited to safe values and are reasonably well balanced, the thickened-edge section has no great advantage over the section of uniform thickness from the standpoint of over-all cost per mile.

12. Transverse joints are required in concrete pavements to relieve warping stresses due to temperature and also to provide for longitudinal expansion and contraction. Longitudinal joints are required to prevent the longitudinal cracking that usually develops otherwise.

13. If proper attention is given to the design of both the slab and joint, the required edge strengthening at joints in thickened-edge slabs can be obtained with a number of the types of load-transfer devices in common use.

14. The thickened-end transverse expansion joint is indicated, both by tests and experience, to be a highly effective method of providing the edge strengthening that is required at transverse joints in thickened-edge slabs.

15. Longitudinal joints of the tongue-and-groove type appear to be considerably more effective than other types in common use in providing the strengthening that is required in the edges of the longitudinal joints of thickened-edge slabs.

Bradbury

In 1938, Bradbury, in his book, *Reinforced Concrete Pavements,* presented tables of stress coefficients (C_c, C_i and C_e), which depended upon values of the variables l/b or a_1/l, for use with Westergaard's maximum load stress equations. Bradbury showed that each of these equations could be expressed as a variable multiplied by a stress coefficient.

Bradbury extended to the complex restrained warping equations his method of using stress coefficients to express long equations in simple form. Since Westergaard's equations often involved hyperbolic functions, rather than normal trigonometric functions, Bradbury's work in this field is especially useful in computing warping stresses.

For a load placed at a corner, Bradbury suggested that the subgrade modulus at, and in the vicinity of, the corner be reduced to ¼ its usual value, since in the Arlington road tests permanent depression of the subgrade was noted. For the maximum load stress along an edge, or in the interior of the slab, Bradbury advocated the use of Westergaard's later equations.

Although Bradbury recommended a complete stress analysis of any pavement, he suggested that in an isolated design case the trial thickness of pavement be determined by means of Older's equation for the corner break, $d = \sqrt{\frac{3P}{\sigma_c}}$ and that a complete stress analysis then be conducted to determine the period of time before the first cracks appear. From this information, the life of the pavement could be estimated. However, Bradbury did not take into account the lateral distribution of traffic on the road.

Pickett

Pickett's equation has the same general shape as Westergaard's theoretical curve but more closely matches the Arlington test data for corner loading. Certain of the equations which have been suggested for determining the maximum load stress for a load applied very close to the corner are repeated below.

Westergaard: $$\sigma_c = \frac{3P}{d^2}\left[1 - \left(\frac{a_1}{l}\right)^{0.6}\right] \qquad \text{(V)}$$

Kelley: $$\sigma_c = \frac{3P}{d^2}\left[1 - \left(\frac{a_1}{l}\right)^{1.2}\right] \qquad \text{(X)}$$

Bradbury: $$\sigma_c = \frac{3P}{d}\left[1 - \left(\frac{a_1}{l\sqrt{2}}\right)^{0.6}\right] \qquad \text{(XII)}$$

Pickett: $$\sigma_c = \frac{4.2P}{d^2}\left[1 - \frac{\sqrt{a_1/l}}{1.1 + 0.185a_1/l}\right] \qquad \text{(IV)}$$

It is believed that the Pickett equation would be the best for analysis if the corner break were the predominant type encountered on our most recently constructed highways. However, in Pennsylvania the most frequent cracks appear to be transverse.

Both magnitude and frequency of high stresses must be considered. The studies of Bradbury (B-4), and data from the Arlington road tests (B-31) suggest that only ⅛ of the total traffic using a lane would be on that lane at a time when the maximum warping stress would be additive to load stresses. Another ⅛ of the traffic could be expected to be on the lane when about ⅔ of the maximum warping stress would be additive to the load stress. For a line of vehicles traveling with outside tires along the edge of the pavement, there will be many more repetitions of the maximum stress due to corner loadings than there will be due to longitudinal edge loadings. However, since the fatigue curve of concrete is not a straight line, the possibility of a few repetitions of a high stress causing failure before many more repetitions of a lower stress will cause failure must be considered.

The greatest edge stresses for heavy wheel loads on slabs 61½ feet long approach a high percentage of the modulus of rupture commonly accepted for the grade of concrete used in pavements (700 psi). A few repetitions of such very high stresses may cause cracks to develop. Reducing slab length to 15 feet would not affect *load* stresses at the interior, edge, or corner but would materially reduce *warping* stresses at the longitudinal edge and interior. The combined stress at the longitudinal edge is then reduced and yet may exceed the corner load stress, but when the *frequency* of stresses is considered, failure may first occur at the corner of a 15-foot slab.

In Pennsylvania at the present time, a slab length of 61½ feet is used. The newer concrete pavements do not seem to fail by the corner break so prevalent before doweled contraction joints were developed and used, but now seem to break initially in a transverse direction because of critical combinations and sufficient repetitions of load and warping stresses along the free longitudinal edge.

As noted above, for very short slabs the corner break may be the first to appear. Because there are no available fatigue data for concrete pavement slabs subjected to localized stresses, a precise comparison of the resistance of a pavement to corner and edge breaks cannot be made. The section across which the corner break will appear is large, so that portion of the slab may also be subject to localized bending stresses.

Others

Loss of subgrade support by the action of pumping pavements has caused cracking and shortened the life of many concrete slabs. A combination of three factors will cause pumping: (a) the presence of free water in the immediate subgrade, (b) a high percentage of fines in the soil at the site, and (c) frequent passes of very heavy loads over the slab. In the absence of any one of these conditions, pumping does not occur. Van Breeman (B-37) stated that 8 inches of special granular material on top of the natural subgrade minimized pumping, reduced damage due to frost action, and increased the load-bearing capacity of pavement slabs.

Giffin (B-8) noted that it is difficult to prevent water from entering the subgrade at joints and recommended the use of granular subbases, both to drain the water from beneath the pavement and to prevent fine material from working its way up through the cracks. He also pointed out that heavy dowels are of value in restricting joint faulting which appears in most cases of pumping joints. Giffin examined many joints on heavily traveled highways of New Jersey and found that dowels had been bent and holes in the concrete enlarged. In many cases the joints had become "frozen," and the dowels could not slip as the slab expanded or contracted. These conditions resulted in cracks 6 to 8 feet from the transverse joints (similar to those which are predominant on many of the Pennsylvania highways subjected to heavy axle loads).

A 1948 report of a committee of the Highway Research Board stated that experience with pavements demonstrated that those constructed without expansion joints developed less pumping than those with expansion joints. Depth of pavement did not appear to affect the pumping action, and the committee stated that no load transfer device effective in preventing pumping had been found.

Experiments are being conducted in several states to determine the need for expansion joints. Anderson, in 1948, after examining the early results of experiments in Michigan and Minnesota, concluded that expansion joints could be eliminated, except for unusual conditions of construction, if the pavement had properly spaced and maintained contraction joints. He advocated spacing contraction joints at 15 feet for aggregates of slag and small siliceous gravel and spacings up to 25 feet for aggregates of crushed granite or limestone. He recommended that spacings greater than 25 feet be avoided, since openings would be of such magnitude that satisfactory seals could not be maintained, and since joints, even with the use of dowels, would be less effective for load transfer. Anderson stated:

> Of the twelve states which place their expansion joints at intervals of 300 feet or more, all employ a contraction joint spacing ranging from between 15 and 30 feet.

Woods (B-42) advocated research to determine the most economic designs of both trucks and pavements. He also stressed the importance of knowledge and treatment of soil conditions. The following is quoted from Wood's paper:

> E. A. Henderson and W. T. Spencer reported important data in regard to 10-yr.-old experimental base course sections on U. S. Highway No. 30 in Indiana. The treatments included eight different base courses, varying in type from bituminous soil mixtures to limestone screenings and dune sands. Thicknesses of treatment ranged from 3 inches to 6 inches. Small amounts of bituminous materials mixed with plastic subgrade soils reduced pavement pumping markedly. The dune sand was particularly effective in eliminating pumping, but the faulting at joints was of measurable and noticeable amounts. This observation, coupled with other performance-survey data showing considerable amounts of faulting on heavily traveled pavements constructed on deep sand-soils, indicates that the sand is being further compacted under the pavement slab by the vibration resulting from repetitions of heavy loads. This being true, precautions need be taken against the use of a deep layer of poorly graded, granular base courses and, in addition, special emphasis must be given to compaction during construction.
>
> As far as rigid pavements are concerned, it appears that heavy duty pavements for truck routes will require either 8 inch to 10 inch slabs with heavy reinforcement, or thinner slabs with less reinforcement, underlain by well-compacted base courses. As an economical expedient in some areas well-graded granular materials and treated bases should be used. The selection of bituminous and concrete overlays to strengthen pavements which are structurally inadequate will probably continue as good design practice; however, experience indicates that moving slabs must be stabilized before overlays can be used satisfactorily.
>
> In regard to flexible pavements, the trend appears to be toward the use of waterbound macadam bases of considerable thickness, or selected granular materials with base and surface courses constructed of bituminous concrete. With the tendency toward rutting, it appears that additional emphasis must be placed on methods and procedures that can be employed during construction to obtain greater compaction of the granular materials. The use of small amounts of liquid bituminous material or cement as binder may partly solve the problem; the development of some method of hardening a claylike soil may be a partial answer to obtaining higher densities in the compaction of base course materials on claylike subgrades.

Woods also pointed out that airport pavement design methods are not applicable to highway design because of the wide differences in repetition of loading and impact. In most airfield construction, slabs of short lengths and widths are used; this construction results in numerous repetitions of corner loadings and relatively few repetitions of free longitudinal-edge loadings.

Road Test One—MD

The major purpose of Road Test One-MD—the Maryland road test—was "to determine the relative effects, on a particular concrete pavement, of four different axle loadings on two vehicle types."[2] A 1.1-mile section of road was closed to regular traffic, and heavy truck loads were applied day and night at a rate which greatly exceeded usual frequencies.

The Portland cement concrete pavement had been constructed in 1941 and had served only light traffic until the beginning of the research tests in 1950. The pavement consisted of two 12-foot lanes of double-parabolic cross-section (7 inches thick at the center line and 9 inches thick at the edges). The reinforcing steel, located from 2 to 3 inches below the top surface, consisted of welded-wire fabric weighing 59.4 pounds per 100 square feet. Transverse joints were at 40-foot intervals; every third one was a ¾-inch expansion joint. Load transfer at transverse joints was obtained by ¾-inch dowels spaced 15 inches center to center. Between the lanes were ⅝-inch tie bars, 4 feet long and spaced 4 feet center to center.

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[2] *Road Test One—MD* (Washington, D.C.: Highway Research Board, 1952), p. 2.

The 1.1-mile strip was divided into two sections with space for a turnaround between the sections. On the south section, two-axle trucks with carefully adjusted 18,000-pound rear-axle loads traveled one lane, and two-axle trucks with 22,400-pound rear-axle loads traveled the other lane. On the north section, travel on one lane was restricted to trucks with a tandem-axle loading of 32,000 pounds, while similar trucks with a tandem-axle loading of 44,800 pounds used the other lane. The drivers followed a prescribed pattern on the test sections; in this way the exact number of load repetitions at points across the slab was known. Around-the-clock operation of the trucks began in June, 1950. By October of that year, the lane subjected to the 44,800-pound tandem-axle loads was cracking at an accelerated rate (about 100 linear feet of cracking per day) and traffic on it was discontinued. Trucks continued to operate on the other three lanes until December of 1950.

Early in 1951, strain and deflection measurements were made on uncracked slabs. These measurements indicated that the greatest stresses occurred when the vehicles were moving, not very rapidly, but very slowly. Prior to these tests, it was believed that if trucks traveled at moderate or high speeds an impact factor greater than 1 was needed. Soil samples were taken from beneath all parts of the four test strips. Sample strips of the concrete slabs had been removed to laboratories for bending fatigue and other tests before the truck loadings began, and other samples were similarly tested after conclusion of truck operation.

The effects of the loadings were compared by reference to the amount (linear feet) of cracking produced in slabs resting on similar types of subgrade soils. The subgrade beneath the south section varied from classification A-1 (Highway Research Board) to classification A-7-6. None of the slabs of the north section rested upon granular subgrades (classifications A-1 or A-2-4). Soils under this section were classified as A-4, A-6, or A-7-8. A comparison of cracking in slabs on A-6 subgrades is given below:

Type of Truck	*Relative Number of Truck Passes to Produce First Crack*
18,000 lb. single-axle	4.2
22,400 lb. single-axle	2.9
32,000 lb. tandem-axle	2.1
44,800 lb. tandem-axle	1

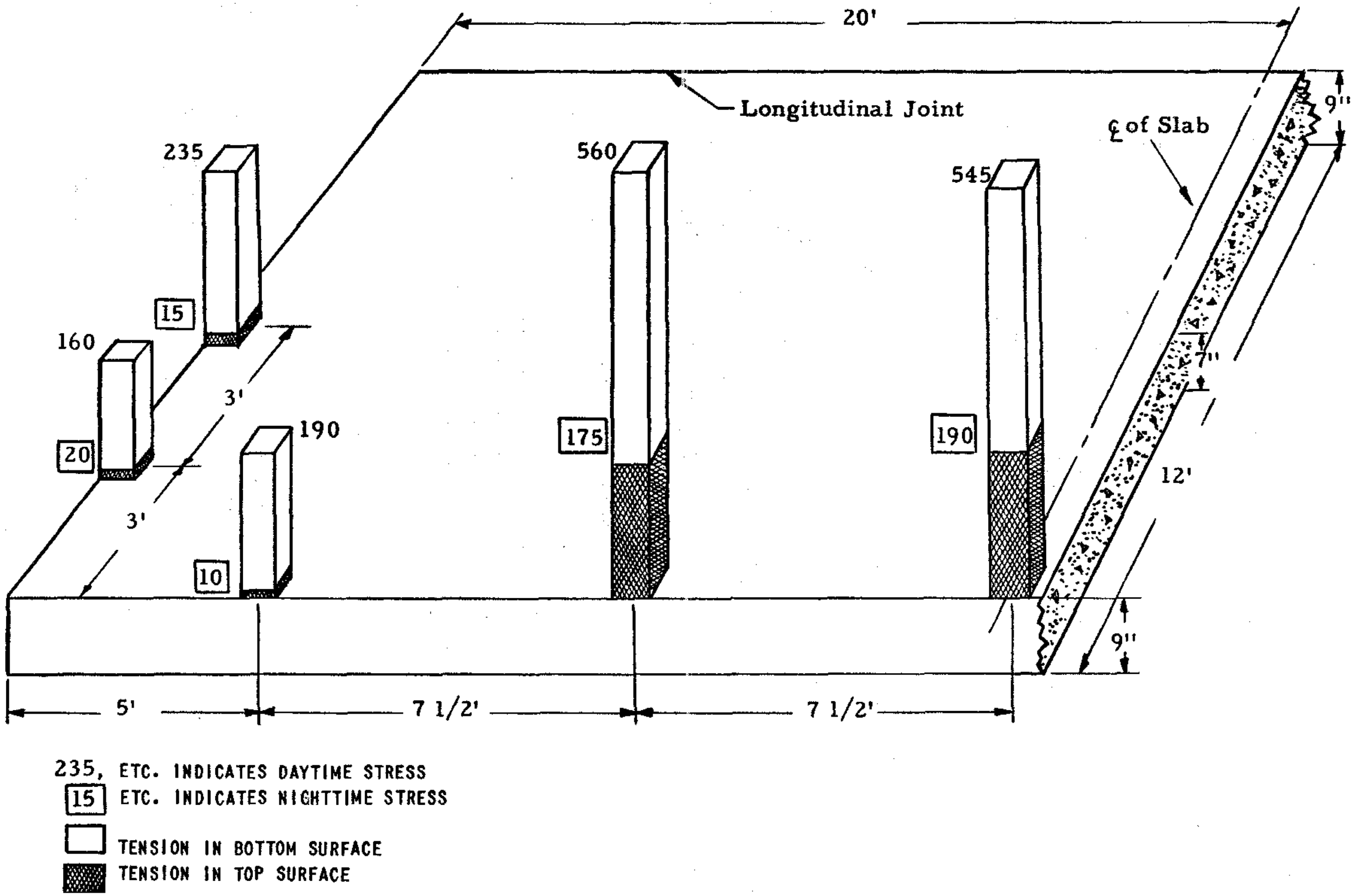
The need for knowledge of lateral distribution of traffic upon a traffic lane becomes apparent from study of the report on Road Test One-MD, which indicates that a wheel centered as much as two feet away from the free longitudinal edge of the pavement produces an appreciable longitudinal-edge load stress.

Restrained temperature warping stresses were also computed from measured strains (see Chart A-I). The slabs did not develop numerous transverse cracks attributable to combined warping and load stresses. The report suggests that a redistribution of stress might occur and that fatigue properties of a concrete slab subjected to localized bending stresses might be much different from those of a concrete beam subjected to bending across its entire width.

Examination of the report indicates that a tandem axle carrying a load twice that of a single axle has more than twice the destructive effect. In addition, it appears that, with tandem axles and very

Chart A-I

MEASURED STRESSES CAUSED BY RESTRAINED TEMPERATURE WARPING ALONG A TRANSVERSE JOINT AND LONGITUDINAL FREE EDGE OF A 12-BY-40-FOOT SLAB



Source: *Road Test One--MD* (Washington: Highway Research Board, 1952), p. 138.

heavy wheel loads placed as much as $4\frac{1}{2}$ feet apart, the load on one wheel affects the load stress in the slab beneath the other.

The change in the strength of the concrete during the tests is shown by the following ranges of modulus of rupture:

At 7 days after pouring (1941)	459-532 psi
Immediately before test (1950)	665-735 psi
After completion of test (1951)	760-915 psi

It appears that some of the loads applied during the test increased the bending strength of the concrete. This concept is not new, for in the past it has been noted that loads strengthen concrete if they produce bending stresses below 50 percent of the modulus of rupture. The compression tests on concrete made immediately before the test traffic was begun showed an average strength of the concrete in direct compression of about 6,900 psi. However, after the test traffic was terminated, the average strength of the cylinders tested was about 6,700 psi. In 1941, shortly after the road was constructed, tests on cylinders showed direct compressive stresses of 4,840 psi.

Appendix C

DERIVATION OF DESIGN METHOD FOR REINFORCED CONCRETE PAVEMENTS

The design method used to determine the optimum thickness of Portland cement concrete pavements is generally similar to that employed by Bradbury (B-4). Other differences aside, the design method employed in this study includes quantification of the effect of the lateral distribution of traffic.

For the three conditions of loading—corner, free longitudinal edge, and interior—the following equations were judged to be most accurate for maximum load stress quantification:

$$\sigma_c = \frac{4.2P}{d^2}\left[1 - \frac{\sqrt{a_1/l}}{1.1 + 0.185a_1/l}\right] \qquad \text{(IV)}$$

$$\sigma_{ex(day)} = 0.57185\frac{P}{d^2}\left[4\log_{10}\left(\frac{l}{b}\right) + 0.3593\right] \qquad \text{(I)}$$

$$\sigma_{ex(night)} = 0.57185\frac{P}{d^2}\left[4\log_{10}\left(\frac{l}{b}\right) + \log_{10}b\right] \qquad \text{(III)}$$

$$\sigma_{ix(night\,or\,day)} = 0.275(1+\mu)\frac{P}{d^2}\left[\log_{10}\left(\frac{Ed^3}{kb^4}\right) - 54.54\left(\frac{l}{L}\right)^2 Z\right] \qquad \text{(VII)}$$

Equation IV is Pickett's equation; Equation I is Westergaard's equation (Poisson's ratio $= 0.15$); Equation III is Kelley's equation (Poisson's ratio $= 0.15$); and Equation VII is Westergaard's revised equation for maximum interior load stress. (Practical use of Equation VII is limited by the type of field tests required.)

Transverse-edge loading will not produce large longitudinal or transverse stresses in the slab unless the pavement has lost subgrade support and pumps. A major premise in this design method is that pumping of pavements is to be eliminated by treatment of the subgrade and addition of a layer of special granular material immediately below the pavement. If pumping is prevented, transverse-edge loading stresses are not critical.

The wheel load (disregarding impact factor) is used in these equations. (Dual wheels are counted as a single wheel.) If data are available showing estimated number of trucks having tandem axles, a ratio may be used to convert tandem-axle wheel loads to single-axle wheel loads.

Restrained temperature warping stresses are considered if the slab is longer than 15 feet. The slab length of 61½ feet now used in Pennsylvania is used in this design method.

Temperature warping stresses in the interior of the slab can be determined by use of Bradbury's curve and his revision of Westergaard's equation:

$$S_{ix} = \frac{Ee\Delta t}{2}\left(\frac{C_x + \mu C_y}{1 - \mu^2}\right) \qquad \text{(IX)}$$

Temperature warping stresses along the free longitudinal edge of the slab can be quantified by use of the equation:

$$S_{ex} = \frac{C_x Ee\Delta t}{2} \qquad \text{(II)}$$

Stresses approaching critical stresses in slabs usually occur through the combination of restrained temperature warping and load stresses. It is difficult to obtain accurate estimates of the maximum load stresses in the interior of the pavement slab, since experimental data are required for each road site. Using average values for the variables L and Z, it was noted that maximum load stresses in the interior of the pavement were substantially below maximum load stresses at the free longitudinal edge. This, together with evidence from highway inspections that transverse cracks begin at the edge of the pavement, led to termination of study of interior stresses.

At the longitudinal edge of the pavement, the temperature warping stresses are additive, during the day, to the load stresses. The maximum temperature differential in the slab in spring and summer is approximately 3°F. per inch of slab thickness. During the fall and winter the maximum temperature differential is about 2°F. per inch. These differentials occur during about six hours of each day (see Chart A-II). By use of the stress coefficient, C_x, and a temperature differential of 3°F. per inch of slab depth, the largest restrained temperature warping stresses were calculated (Equation II).

In pavement design, it is necessary to estimate the number and magnitude of combined stresses. This requires estimates of the effects of wheel loads of different weights. Table A-4 shows load-distribution radii for selected wheel loads, and these, together with other pertinent data, are used in the computation of maximum load stresses at the longitudinal edge of the pavement. Table A-5 illustrates determination of maximum load stresses (column 9), restrained temperature warping stresses (column 12), and combined load and warping stresses (column 13).

Table A-4
LOAD-DISTRIBUTION RADII FOR SELECTED WHEEL LOADS

	Load-distribution Radius—a (inches)— at Specified Locations of Wheel on Slab			
Wheel Load (pounds)	*Corner*	*Interior*	*Transverse Edge*	*Longitudinal Edge*
(1)	(2)	(3)	(4)	(5)
1,000	2.5	2.5	3.5	3.5
2,000	3.2	3.2	4.5	4.5
3,000	3.9	3.9	5.0	5.6
4,000	4.6	4.6	5.5	6.7
5,000	5.0	5.2	6.1	7.4
6,000	5.3	5.7	6.6	8.0
7,000	5.6	6.1	7.0	8.6
8,000	5.8	6.5	7.4	9.1
9,000	6.0	6.9	7.8	9.6
10,000	6.3	7.3	8.1	10.1
11,000	6.5	7.7	8.5	10.5
12,000	6.6	8.0	8.8	11.0
13,000	6.8	8.3	9.1	11.4
14,000	7.0	8.6	9.4	11.8
15,000	7.2	8.9	9.7	12.1

Table A-5

COMPUTATION OF COMBINED LONGITUDINAL-EDGE STRESS—DAY*

Wheel Load (pounds) P	*Pavement Thickness (inches)* d	*Load Radius (inches)* a	a/d	*Pressure Distribution Radius (inches)* b	*Radius of Relative Stiffness* l	l/b	*Bradbury's Coefficient (Longitudinal Free Edge)* C_e	*Load Stress* $\sigma_{ex} = \frac{P}{d^2}C_e$	B/l	C_x	*Restrained Temperature Warping Stress* $S_{ex} = \frac{C_x E e \Delta t}{2}$	*Combined Longitudinal-edge Stress* $\sigma_{ex} + S_{ex}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
3,000	8	5.6	0.70	5.29	30.6	5.79	1.95	91	24.1	1.04	251	342
5,000	8	7.4	0.92	6.82	30.6	4.49	1.70	133	24.1	1.04	251	384
7,000	8	8.6	1.07	8.14	30.6	3.76	1.52	166	24.1	1.04	251	417
9,000	8	9.6	1.20	9.14	30.6	3.35	1.41	198	24.1	1.04	251	449
10,800	8	10.4	1.30	10.00	30.6	3.06	1.32	222	24.1	1.04	251	473
12,000	8	11.0	1.37	10.70	30.6	2.86	1.25	235	24.1	1.04	251	486
13,200	8	11.5	1.44	11.23	30.6	2.72	1.20	248	24.1	1.04	251	499
14,400	8	11.9	1.49	11.67	30.6	2.62	1.16	262	24.1	1.04	251	513

*Where: $k = 200$
$E = 4{,}000{,}000$ psi
$\mu = 0.15$
$e = 0.000005$
$\Delta t = 3°F./inch$
Slab Length $\geq$ 45 feet
Slab Width = 12 feet

Chart A-II

EXAMPLES OF DAILY AND SEASONAL TEMPERATURE VARIATIONS IN CONCRETE PAVEMENT SECTIONS: ARLINGTON ROAD TEST

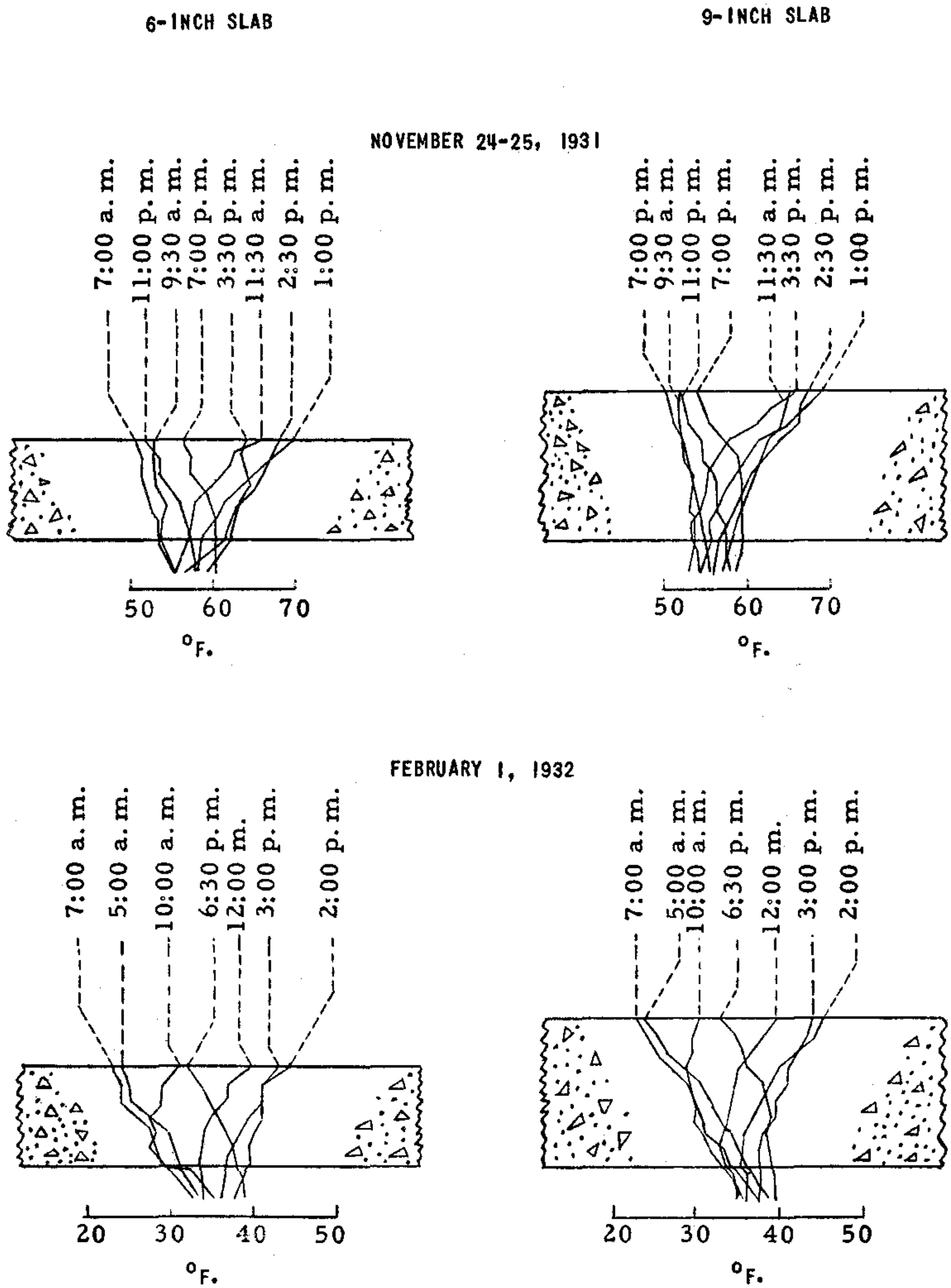
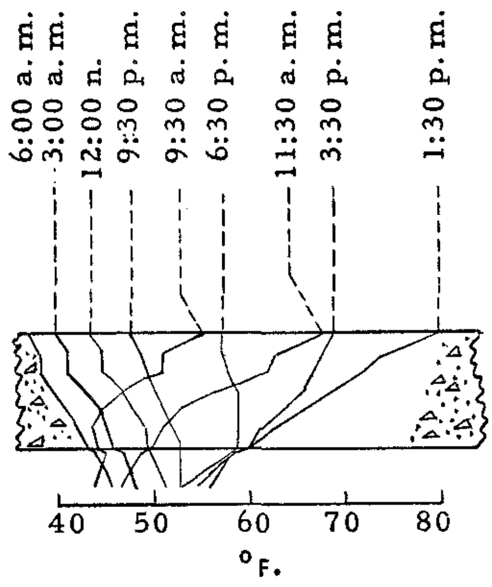
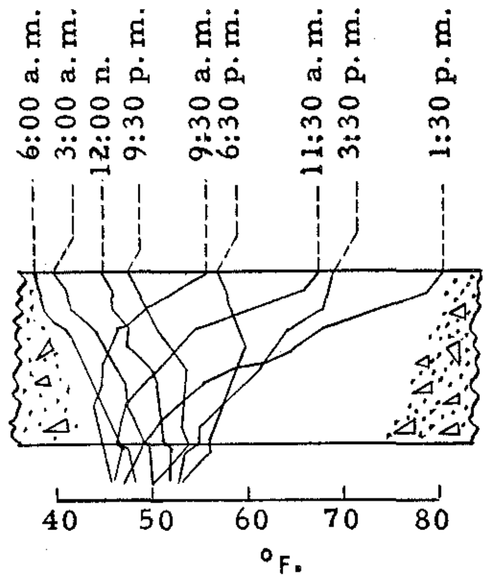


Chart A-II *(Continued)*

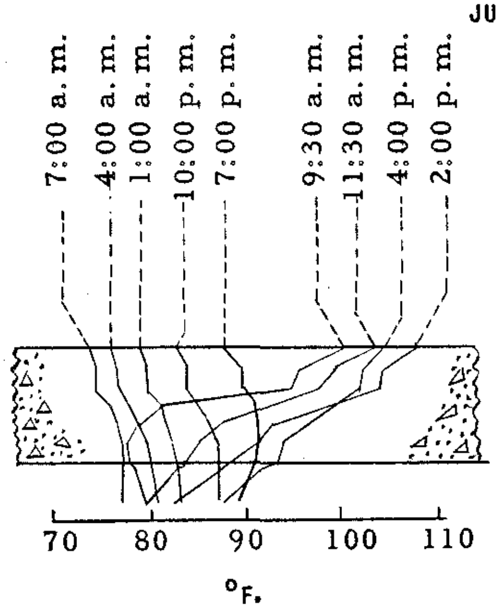
6-INCH SLAB | 9-INCH SLAB

APRIL 14-15, 1932





JULY 12-13. 1932



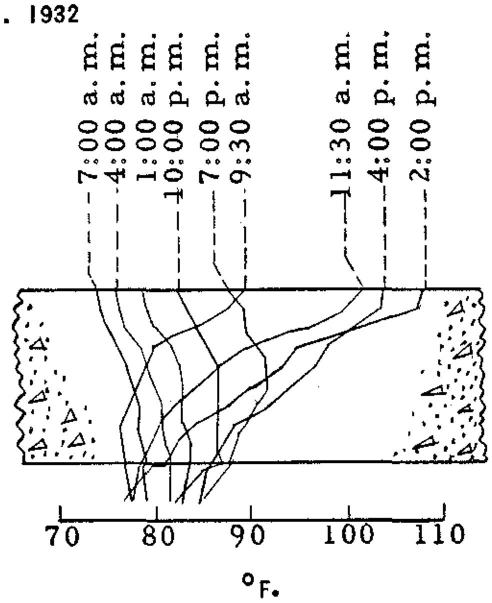


Table A-6

CALCULATION OF EQUIVALENT 5,000-POUND WHEEL-LOAD REPETITIONS*
FOR $\Delta' t = 3°$ AND $\Delta' t = 2°$

I. $\Delta' t = 3°$

Wheel Load (pounds)	*Wheel Position*†	Δt‡	*Combined Longitudinal Edge Stress (psi)*	*Percent of Modulus of Rupture*	*Repetition to Cause Failure*	*Estimated Wheel-position Repetitions per Year*	*Equivalent 5,000-pound Repetitions per Year*§
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
3,000.........	a	24°	below 350	...	Infinite	...	0
4,000.........	a	24°	363	51.8	550,000	127	39
5,000.........	a	24°	384	54.8	170,000	61	61
	b	24°	357	51.0	850,000	156	31
6,000.........	a	24°	402	57.4	65,000	78	204
	b	24°	372	53.1	340,000	200	100
7,000.........	a	24°	418	59.8	30,000	78	442
	b	24°	384	54.8	170,000	199	199
	c	24°	351	50.1	Infinite	...	0
8,000.........	a	24°	433	61.9	17,000	74	740
	b	24°	396	56.6	90,000	191	361
	c	24°	360	51.4	700,000	393	95
9,000.........	a	24°	448	64.1	8,000	74	1575
	b	24°	408	58.3	48,000	190	673
	c	24°	369	52.7	400,000	390	166
10,000.........	a	24°	463	66.2	3,900	36	1570
	b	24°	421	60.1	28,000	92	553
	c	24°	378	54.0	234,000	189	137
11,000.........	a	24°	473	67.6	2,600	9.1	595
	b	24°	428	61.2	20,000	23.4	199
	c	24°	384	54.8	170,000	48.2	48.2
12,000.........	a	24°	487	69.5	1,500	1.4	159
	b	24°	440	62.8	12,000	3.7	52.4
	c	24°	392	56.0	110,000	7.6	11.7
13,000.........	a	24°	496	70.9	1,040	0.6	102
	b	24°	447	63.9	8,500	1.5	30
	c	24°	398	56.9	80,000	3.1	6.6
15,000.........	a	24°	521	74.4	320	0.3	154
	b	24°	468	66.9	3,150	0.7	37.8
	c	24°	413	59.1	35,000	1.5	7.3
	d	24°	358	51.2	750,000	2.7	0.6

Table A-6 (Continued)

II. $\Delta' t = 2°$

Wheel Load (pounds)	*Wheel Position*†	Δt‡	*Combined Longitudinal Edge Stress (psi)*	*Percent of Modulus of Rupture*	*Repetition to Cause Failure*	*Estimated Wheel-position Repetitions per Year*	*Equivalent 5,000-pound Repetitions per Year*§
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
9,000.........	a	16°	365	52.1	500,000	74	25.2
10,000.........	a	16°	380	54.3	210,000	36	29.2
11,000.........	a	16°	390	55.7	120,000	9.1	12.9
12,000.........	a	16°	404	57.7	60,000	1.4	4.0
13,000.........	a	16°	413	59.0	37,000	0.6	2.8
	b	16°	365	52.0	450,000	1.5	0.6
15,000.........	a	16°	438	62.6	13,000	0.3	3.9
	b	16°	383	54.7	180,000	0.7	0.6

*Slab Depth = 8 inches; Slab Length > 45 feet; Slab Width = 12 feet; k = 200 psi/inch. Wheel-load distribution for which equivalents are illustrated appears in Table A-8, column 2.

†Wheel positions are as follows:

(a) Center of wheel 6 inches from longitudinal edge

(b) Center of wheel 12 inches from longitudinal edge

(c) Center of wheel 18 inches from longitudinal edge

(d) Center of wheel 24 inches from longitudinal edge

‡Temperature differential of 3°F. per inch slab depth—spring and summer; temperature differential of 2°F. per inch slab depth—fall and winter.

§Repetitions of 5,000-pound wheel loads placed at position (a) on the slab.

Frequencies of stress combinations are of importance. Since the temperature differential of 3°F. per inch of slab depth occurs about ½ of the days of the year and will be of that magnitude for approximately six hours per day, only about ⅛ of the yearly traffic on a lane occurs when load stresses are additive to the greatest warping stresses. Another ⅛ of the yearly traffic occurs when warping stresses caused by a temperature differential of 2°F. per inch of slab thickness are additive to the load stresses. These data were used in estimating numbers of repetitions of combined load and warping stresses.

In almost all design methods for concrete pavement thickness, the effect of lateral distribution of vehicles upon number of stress repetitions is not taken into account. Location of the outside wheels of vehicles on the pavement varies. Taragin, in 1945, noted the average lateral placement of commercial vehicles on lanes of different widths. Extracts of these data appear in Tables 10 and 11 (Section II).

Since the number of wheel loads at different points across the slab could be estimated, the longitudinal-edge stress or corner stress produced by a load away from the edge or corner could be computed. By combining data from Table 11 (percent of total traffic at different points across the road) with information on load stress effects (from Road Test One-MD), the numbers of repetitions of stresses at the longitudinal edge or near the corner were estimated. Column 7 of Table A-6, Estimated Wheel-Position Repetitions per Year, reflects this method.

Corner and longitudinal-edge loadings, taking into account lateral distribution of vehicles on the lane, were analyzed. When corner loadings, only, were considered, there appeared to be virtually no limit to the estimated life of pavements, since corner loadings produce stresses of relatively low magnitudes. However, when longitudinal-edge loadings were considered, estimated life agreed well with observed life. Further analysis was limited to longitudinal-edge stresses.

A pavement may be designed for a specific number of repetitions of a basic wheel load. The wheel load used in this design method was 5,000 pounds. It was therefore necessary to express other wheel loads and warping effects in terms of repetitions of this basic load. Bradbury's fatigue curve for concrete in flexure was used to make the conversion. Stress repetitions were reduced to equivalent repetitions of a single 5,000-pound load placed at the edge of the slab during the day, with a temperature differential of 3°F. per inch. Table A-6 illustrates this method. Table A-7 is a summary, for all wheel positions, of equivalent 5,000-pound wheel-load effects for slab widths of 9, 10, 11, and 12 feet.

Table A-7

SUMMARY OF 5,000-POUND WHEEL-LOAD EQUIVALENTS FOR SLAB WIDTHS OF 9, 10, 11, AND 12 FEET*

(I.) $\Delta t = 3°$/inch of slab depth

Wheel Load (pounds)	*Slab Width*			
	12 feet	*11 feet*	*10 feet*	*9 feet*
(1)	(2)	(3)	(4)	(5)
4,000	39	89	323	875
5,000	92	204	706	1,675
6,000	304	677	2,340	5,553
7,000	1,141	1,430	4,950	11,860
8,000	1,196	2,667	8,929	20,756
9,000	2,414	5,397	18,255	42,980
10,000	2,260	5,071	17,248	41,514
11,000	842	1,889	6,448	15,618
12,000	223	501	1,715	4,160
13,000	139	311	1,071	2,627
15,000	200	450	1,563	3,890

(II.) $\Delta t = 2°$/inch of slab depth

Wheel Load (pounds)	*Slab Width*			
	12 feet	*11 feet*	*10 feet*	*9 feet*
(1)	(2)	(3)	(4)	(5)
9,000	25	58	209	565
10,000	29	67	242	655
11,000	13	30	107	289
12,000	4	9	33	90
13,000	3	8	28	69
15,000	5	10	36	94

*Slab depth = 8″; Slab length $\geq$ 45′; k = 200 psi/in. Wheel-load distribution for which load equivalents are illustrated appears in Table A-8, column 2.

Table A-8

EMPIRICAL EVALUATION OF EQUIVALENT WHEEL-LOAD PERCENT FACTORS AND EXPERIMENTAL VERIFICATION OF FACTORS FOR SLABS 12 FEET WIDE*

		12-foot Slabs			11-foot Slabs		10-foot Slabs		9-foot Slabs	
Wheel Load (pounds)	*Estimated Repetitions per Year*†	*Equivalent 5,000-pound Loads*	*Empirical Equivalent Wheel-Load Percent Factors*	*Equivalent Wheel-Load Percent Factors*	*Equivalent 5,000-pound Loads*	*Empirical Equivalent Wheel-Load Percent Factors*	*Equivalent 5,000-pound Loads*	*Empirical Equivalent Wheel-Load Percent Factors*	*Equivalent 5,000-pound Loads*	*Empirical Equivalent Wheel-Load Percent Factors*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
4,000 ...	145,000	39	0.03	0.05	89	0.06	323	0.22	875	0.60
5,000 ...	69,200	92	0.13	0.13	204	0.30	706	1.02	1,675	2.42
6,000 ...	89,000	304	0.34	0.39	677	0.76	2,340	2.60	5,553	6.20
7,000 ...	88,600	641	0.73	0.77	1,430	1.60	4,950	5.60	11,860	13.40
8,000 ...	85,000	1,196	1.40	1.50	2,667	3.10	8,929	10.50	20,756	24.40
9,000 ...	84,200	2,439	2.90	2.90	5,455	6.50	18,464	21.90	43,545	51.70
10,000 ...	40,750	2,289	5.60	5.20	5,138	12.60	17,490	42.90	42,169	105.00
11,000 ...	10,400	855	8.20	8.20	1,919	18.40	6,555	63.00	15,907	153.00
12,000 ...	1,642	227	13.80	13.60	510	31.00	1,748	107.00	4,250	258.00
13,000 ...	660	142	21.50	23.60	319	48.40	1,099	165.00	2,696	407.00
15,000 ...	330	204	61.80	63.30	460	139.00	1,599	484.00	3,984	1,210.00
Total ...		8,428			18,868		64,203		153,270	

*k = 200 psi/in.
Slab Depth = 8 inches
Slab Length $\geq$ 45 feet
†Wheel-load distribution used for empirical evaluation of factors.

Derivation of equivalent wheel-load percent factors is illustrated in Table A-8 by reference to a specific wheel-load distribution. (This distribution is, of necessity, the same as that used as a basis for the example contained in Table A-6.) The magnitudes of equivalent wheel-load percent factors (number of wheel loads of a specific weight distributed normally across the pavement needed to produce the stress effect of a 5,000-pound edge load) are functions of lateral placement and wheel-load conversion factors. Consequently, the magnitudes of equivalent wheel-load percent factors are constant for specific depths of slab and subgrade moduli, and computations shown in Tables A-6, A-7, and A-8, for a specific load distribution, represent an empirical verification of the method.

Differences in pavement life resulting from differences in lane widths may be illustrated by dividing number of equivalent wheel-load repetitions which cause failure (estimated fatigue point) by the total numbers of equivalent load repetitions per year (see Table A-8):

12-foot lanes	170,000 ÷ 8,428 = 20.2 years
11-foot lanes	170,000 ÷ 18,868 = 9.0 years
10-foot lanes	170,000 ÷ 64,203 = 2.6 years
9-foot lanes	170,000 ÷ 153,270 = 1.1 years

These, and similar computations, were used as a basis for determination of factors of reduction in pavement life associated with reduced pavement width.

LIST OF SYMBOLS

σ_c = Maximum unit stress due to vertical load on pavement at distance a_1 from the corner. (psi)

σ_{ex} = Maximum longitudinal unit stress due to vertical load at point six inches from free longitudinal edge and at least five feet from transverse edge. (psi)

σ_{ix} = Maximum longitudinal unit stress due to vertical load at point at least three feet from longitudinal edge and at least five feet from transverse edge. (psi)

S_{ex} = Maximum longitudinal unit stress due to Δt six inches from free longitudinal edge. (psi)

S_{ix} = Maximum longitudinal unit stress due to Δt in interior. (psi)

P = Vertical wheel load. (pounds)

d = Pavement thickness. (inches)

E = Modulus of elasticity of concrete. (psi)

μ = Poisson's ratio.

k = Modulus of subgrade reaction. (psi/in)

l = $\sqrt[4]{\frac{Ed^3}{12(1-\mu^2)k}}$ = radius of relative stiffness. (inches)

a = Equivalent wheel-contact-area circle radius. (inches)

a_1 = $a\sqrt{2}$ = distance from slab corner along bisector of corner angle to center of area of load—corner loading condition. (inches)

b = radius of circle of equivalent distribution of pressure. (inches)
$b = \sqrt{1.6\,a^2 + d^2} - 0.675d$ when $a < 1.74d$.
$b = a$ when $a > 1.74d$.

L = Maximum value of radius of circular area within which redistribution of subgrade reactions is made. The center of this circle is at point of load application. (inches)

Z = Ratio of reduction of maximum deflection.

C = Stress coefficient.

C_c = Bradbury's stress coefficient for maximum load stress due to corner loading.

C_e = Bradbury's stress coefficient for maximum load stress due to longitudinal free edge loading.

C_i = Bradbury's stress coefficient for maximum load stress due to interior loading.

e = Coefficient of thermal expansion of concrete. (°F.)

Δt = Temperature differential between the top and bottom of pavement slab. (°F./in.)

C_x = L_x/l = a coefficient.

C_y = L_y/l = a coefficient.

L_x = Length of slab. (inches)

L_y = Width of slab. (inches)

B = Free length (or width) of slab. (inches)

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