A PRACTICAL METHOD FOR THE DESIGN OF PIT ROADS

by

James L. Post

A Thesis Submitted to the Faculty of the

DEPARTMENT OF MINING AND METALLURGY

In Partial Fulfillment of the Requirements
For the Degree of

MASTER OF SCIENCE

In the Graduate College

THE UNIVERSITY OF ARIZONA

1 9 6 3

STATEMENT BY AUTHOR

This thesis has been submitted in partial fulfillment of requirements for an advanced degree at the University of Arizona and is deposited in the University Library to be made available to borrowers under rules of the Library.

Brief quotations from this thesis are allowable without special permission, provided that accurate acknowledgment of source is made. Requests for permission for extended quotation from or reproduction of this manuscript in whole or in part may be granted by the head of the major department or the Dean of the Graduate College when in his judgment the proposed use of the material is in the interests of scholarship. In all other instances, however, permission must be obtained from the author.

SIGNED: James L. Post

APPROVAL BY THESIS DIRECTOR

This thesis has been approved on the date shown below:

H E Krumlauf
H. E. KRUMLAUF
Professor of Mining Engineering

January 8, 1963
Date

ACKNOWLEDGMENTS

The author wishes to express his appreciation to E. R. Drevdahl, Associate Professor of Mining Engineering, and to H. E. Krumlauf, Professor of Mining Engineering, for their assistance and constructive criticism during the preparation of this manuscript. The author is also grateful to the County of Los Angeles Road Department for the aid and assistance which was given in the research and preparation of this thesis.

TABLE OF CONTENTS

		Page
LIST OF TABLES		vi
LIST OF ILLUSTRATIONS		vii
ABSTRACT		ix
CHAPTER		
1.	THE DEVELOPMENT OF ROADBED DESIGN PROCEDURE	1
	Pavement Types	1
	Review of Flexible Pavement Design Methods	2
	Review of Rigid Pavement Design Methods	3
	Review of the California Bearing Ratio Soil Test	4
2.	HEAVY-DUTY TRUCK HAULAGE	6
	Truck Classification	6
	Legal Dimensions	7
	Equivalent Wheel Load Design	7
3.	ROADBED CLASSIFICATION AND MATERIAL STANDARDS	10
	Roadway Surface Types	10
	Roadway Service Life	11
	Roadbed Structural Members	12
	Aggregate Base Material	13
4.	ROADBED CONSTRUCTION COSTS	15
	Cost Factors	15
	Comparative Pavement Analysis	15
	Cost Index	16
	Flexible Versus Rigid Pavements	16
	Continuous Reinforcement	20
	Maintenance Costs	21
5.	TRUCK PERFORMANCE	23
	Factors Affecting Performance	23
	Rolling Resistance	23
	Traction Loss	25
	Tire Service	25

Page

CHAPTER

6. FLEXIBLE PAVEMENT DESIGN 38

Design Factors 38
Equivalent Single-Wheel Load 41
Load Repetition 43
Gravel Equivalents 45
Sample Design Computations 47
Treated Base 49

7. RIGID PAVEMENT DESIGN 51

Design Considerations 51
Subgrade Modulus 52
Equivalent Single-Wheel Load 52
Safety Factor 59
Design Curves 60
Subbase Requirements 66
Sample Design Computations 68
Dowels 69

8. CONCLUSIONS 76

Summary 76
Further Research 77

BIBLIOGRAPHY 78

LIST OF TABLES

TABLE		Page
I.	DESIGN AND COST ASSUMPTIONS	17
II.	COMPARATIVE PAVEMENT ANALYSIS - FLEXIBLE PAVEMENT	18
III.	COMPARATIVE PAVEMENT ANALYSIS - RIGID PAVEMENT	19
IV.	APPROXIMATE ROLLING RESISTANCE	24
V.	APPROXIMATE COEFFICIENTS OF TRACTION	26
VI.	TIRE LIFE EXPECTANCY TABLES	27
VII.	TIRE RIMS AND DUAL-WHEEL SPACING	36
VIII.	COHESIOMETER VALUES	46
IX.	RADIUS OF RELATIVE STIFFNESS	56
X.	REQUIRED SUBBASE THICKNESS IN INCHES	66

LIST OF ILLUSTRATIONS

Page

FIGURE

1. Tire prices for heavy-duty off-highway trucks 28

2. Tires for earth-moving and mining service for short hauls. Maximum speed - 30 mph 29

3. Tires for earth-moving service for short hauls. Maximum speed - 10 mph 30

4. Wide-base tires for earth-moving service for short hauls. Maximum speed - 30 mph 31

5. Wide-base tires for earth-moving service for short hauls. Maximum speed - 10 mph 32

6. Load carrying capacity of heavy duty tires. Dashed lines indicate extrapolated values 34

7. Radius of contact area for various wheel loads 37

8. Original flexible pavement design curves 40

9. Design curves for flexible pavements 42

10. Approximate relationship between the California Bearing Ratio and the modulus of subgrade reaction 53

11. Chart for determination of equivalent single-wheel load for rigid pavements and dual-wheel gear 55

12. Chart for determination of equivalent single-wheel load for rigid pavements and dual-tandem gear 58

13. Fatigue of concrete in flexure 60

14. Design chart for portland-cement concrete pavements having protected corners and subgrade modulus of 50 psi/inch 61

15. Design chart for portland-cement concrete pavements having protected corners and subgrade modulus of 100 psi/inch 62

16. Design chart for portland-cement concrete pavements having protected corners and subgrade modulus of 200 psi/inch 63

Page

FIGURE

17. Design chart for portland-cement concrete pavements having protected corners and subgrade modulus of 400 psi/inch 64

18. Effect of various subbase thicknesses on 'k' value . . . 67

19. Determination of total capacity factor of a dowel system 71

20. Determination of correction factors for the load-transfer capacity of dowels 72

21. Load-transfer capacities of single dowels for joint openings of various widths 73

ABSTRACT

Pavement design methods, heavy-duty truck haulage problems, and roadbed structural design standards are discussed in order to delineate the requisites which must be satisfied in designing pit haulage roads.

Methods of cost estimation for the construction of roads designed for heavy-duty service are shown, with particular reference to the comparison between equivalent flexible type and rigid type pavements, and the factors of truck performance which necessitate the construction of the more permanent high-type roads are discussed.

Practical methods for the design of heavy-duty flexible type pavement and rigid type pavement are developed with examples which show how these design methods may be used.

CHAPTER 1

THE DEVELOPMENT OF ROADBED DESIGN PROCEDURES

Pavement Types. Roadbed design procedures have been developed by two parallel methods of consideration, the theoretical and the empirical methods. The theoretical method of design has been altered many times in order to attempt to explain the results of actual road tests which have been undertaken.

The term 'roadbed' and the term 'pavement' will be used synonymously in this manuscript since embankment material will not be considered. The roadbed ordinarily consists of the pavement and any embankment material which is necessary to construct the road. Embankment material is often referred to as fill.

There are two main types of pavements, the flexible pavement and the rigid pavement. There is also a third type of pavement which is sometimes referred to as semi-flexible pavement (1).

Flexible type pavements carry their loads by granular interaction and their design is predicated on the condition of each layer being of sufficient thickness to not overstress the underlying layers.

Rigid type pavements carry their loads by slab strength and the strength of the subgrade is seldom the controlling factor. Design is based on elastic theory of continuously supported slabs, and is predicated on maintaining tensile stresses in the slab within the allowable flexural strength of the slab material.

Pavements constructed of soil-cement, lime-clay, asphalt stabilized bases, and many other materials are neither wholly rigid nor flexible. They possess considerable slab strength, and at the same time deflect sufficiently to transmit sizeable stresses to the subgrade. The design, in the case of such semi-flexible type pavements, may be governed by the strength of either the subgrade or that of the pavement material.

Review of Flexible Pavement Design Methods. Thickness design methods for flexible pavements are based on research done by Boussinesq (2), which was published in 1885. From this research a relationship was developed for the vertical stress at any point on a horizontal plane due to a concentrated load acting on a semi-infinite elastic body.

The Boussinesq equation is a simplification of actual roadbed conditions. Stress distribution under a flexible pavement is extremely complex. Vertical stresses resulting from other types of loading have been obtained from the Boussinesq equation.

A further development was made in pavement design by Burmister (3). His research results were published in 1943. Burmister extended the Boussinesq equation to include two layers, based upon the assumptions: the two layers are elastic bodies; the surface layer is weightless, of infinite extent horizontally but of finite depth, and the second layer is infinite in extent both horizontally and vertically; the interface between pavement and subgrade is continuous and always in contact.

The principal criticism of the Burmister theory is that the elements of a flexible pavement are assumed to function as perfectly elastic materials. The actual behavior of the materials is never perfectly elastic.

The roads in use today were designed primarily by empirical methods, i.e., by the testing of roads under similar conditions over a number of years. Many methods of calculating the required thickness of flexible pavement have been proposed during the last 62 years. These methods have been based upon both theoretical and empirical studies. The first published method of calculating the thickness of flexible pavements is the so-called Massachusetts Rule of 1901. By 1944 more than 22 methods of design of flexible pavement had been published (4).

There have been many experimental test roads constructed in order to justify existing roadbed design procedures, or to consider new design procedures. The most important road tests undertaken were the WASHO (Western Association of State Highway Officials) test (5), from 1952 to 1954, and the AASHO (American Association of State High Officials) test (6), from 1958 to 1960. The AASHO road tests were undertaken at a cost of more than twenty-seven million dollars.

Review of Rigid Pavement Design Methods. Thickness design methods for rigid pavement are also based on the research done by Boussinesq and Burmister. However, the use of a rigid slab of pavement, which is elastic within certain limits, permits the development of a theoretical roadbed design procedure which closely approximates empirical test results.

A level of approach, first applied to pavement analysis by Westergaard (7), treats the pavement as a plate on an elastic (set of independent springs) foundation. The Westergaard equations have been extended to permit determination of subgrade stresses. This is done by replacing the set of independent elastic springs (or equivalently, a dense liquid) by an elastic solid. It should be noted that each of the previously mentioned levels of approach has been applied with the assumption that the pavement and subgrade are elastic materials.

The first experimental rigid pavement test road was the Bates Experimental Road (8) constructed in 1926. Many rigid pavement test roads have been constructed since then. Controlled tests are still being conducted on many of these test roads.

The most important rigid pavement test road which has been constructed is, again, the AASHO test road. The test results from this road are presently being compiled and studied. The Highway Research Board has released several publications concerning this test. The road was constructed of dual roadway sections of rigid pavement and of flexible pavement. The varied roadways were constructed so that a comparison of the equivalent sections of rigid and flexible pavements could be made.

Review of the California Bearing Ratio Soil Test. The CBR (California Bearing Ratio) method of thickness design (9) was developed by the California Division of Highways about 1930. The CBR method was used in California until it was superseded by the Hveem Stabilometer 'R' value method (10) in 1951.

The CBR is a comparative measure of the shearing resistance of a soil. It is used with empirical curves to design asphalt pavement structures. This test consists of measuring the load, in pounds per square inch, required to force a piston of standard size into a soil specimen a certain depth at a specified rate (9, 11). The test result is expressed as a percentage of the load required to force the piston the same depth into a standard sample of crushed rock. The soil specimens are compacted to maximum density and soaked to achieve maximum saturation. The CBR method, with its numerous variations, is probably the most widely used method of designing asphalt pavement.

The Corps of Engineers investigated the CBR design procedure along with the other common design procedures extant in 1942. A series of airfield experimental runway sections were constructed and tested to destruction (12). The CBR method of thickness design for flexible pavements was chosen on the basis of reliability and consistency of soil tests, time required for tests, and convenience and flexibility of the testing method.

CHAPTER 2

HEAVY-DUTY TRUCK HAULAGE

Truck Classification. The trucks which are designed for off-highway heavy-duty service differ from regular commercial trucks primarily in size. The heavy-duty truck is fabricated to hold a larger volume and to carry a heavier load.

A general classification of truckshas been made on the basis of wheel and axle spacing (13). This classification is used in many states to aid in the design of flexible pavements (14). Trucks are classified according to the number of axles which are used to support the truck. Special tag-along axles are not considered and, generally, two-axle single-tire vehicles are excluded from the truck list.

Trucks are classified as 2-axle, 3-axle, 4-axle, 5-axle, and 6-axle vehicles. The percentage of trucks of each classification varies in different parts of the United States. As an example, the truck count breakdown in Los Angeles County, California, is about 80% 2-axle, 10% 3-axle, 4% 4-axle, 4% 5-axle, and 2% 6-axle.

The predominant majority of the heavy-duty off-highway trucks are 2-axle and 3-axle vehicles. Most of these trucks are single-unit 2-axle vehicles. There are some fairly comprehensive listings published annually which show the many types of trucks available for purchase (15).

Legal Dimensions. Each state of the United States has enacted legislation regulating the use of highways within the state. Although these laws vary from one state to another they are generally quite uniform. The average truck size, weight, and speed restrictions are as follows: The maximum permissible width is 96 inches, the height is 12.5 feet, and the length is 55 - 60 feet. The maximum axle load is 18,000 pounds, the maximum tandem axle load is 32,000 pounds, and the maximum gross weight is 60,000 pounds. The speed limit is 45 - 55 miles per hour (8,16).

The truck designed for off-highway heavy-duty service is not intended to be used on a highway. A highway is a public roadway. The new larger trucks cannot even be delivered via public highways, but must be shipped by rail in parts and assembled at the site of operations. There are several reasons for this restriction. Some dimensions of one of the new large trucks follow (17). The vehicle tare weight is 63 tons, the payload is 100 tons, the width is about 14 feet, and the height is more than 15 feet. The truck is supported by three single axles.

The modern highway primary arterial structures are constructed to a load design rating of H20-S18. This means a maximum gross truck load of 20 tons and a trailer load of 18 tons (18). This load limitation, along with the previously mentioned highway load limitations, means that the new large trucks, empty, are too heavy and too large to be allowed on a highway.

Equivalent Wheel Load Design. In order to design roads on the basis of truck usage it is necessary to develop an arbitrary weight constant for one wheel load, or one axle load. This is done so that the effects of different weight loading may be compared. Different weight constants have been chosen in different states, but the most common constant is the 5000 pound wheel load. This loading is referred to as the EWL (Equivalent Wheel Load) constant (14).

The EWL constant was first used in the development of a road design method in California (19) about 1942. Trucks with different numbers of axles were compared with respect to their destructive effect on test roads and highways. A series of statewide traffic counts were made and the trucks were counted and classified according to the number of axles per truck. On the basis of these data EWL constants were assigned to trucks according to the number of axles per truck.

Annual traffic counts are made in California to observe highway vehicle usage. A modification was made in the EWL constants for trucks in 1957 due to the destructive effects of modern trucks on highways (10). These EWL constants for the number of trucks using a road are compiled to compute a TI (Traffic Index) rating. This TI rating is then used to aid in the design of a road which is structurally adequate to carry the commercial traffic of this area.

In the design of pit roads this Traffic Index cannot be used since the EWL constants used by a state bear little relationship to the type of trucks which may use a pit road. However, it has been shown (20) that Equivalent Wheel Loads may be related directly to axle loads. It may be convenient to compare the relative destructive effects of different trucks when considering haulage problems and pavement design. This relationship is given by the equation

$N_1(2)^{P_1} = N_2(2)^{P_2}$ where N_1 and N_2 are the number of axles carrying, respectively, the axle loads (in tons) P_1 and P_2. In a given case, assuming that values for P_1, P_2, and N_2 are known, the equivalent number of coverages N_1 can be computed from the expression $N_1 = N_2(2)^{P_2 - P_1}$. The EWL constant is, again, one 5000 pound wheel load, or one 5 ton axle.

CHAPTER 3

ROADBED CLASSIFICATION AND MATERIAL STANDARDS

Roadway Surface Types. In order to study and compare the life characteristics of rigid and flexible pavements, roadways are classified according to surface types. This classification also aids in making an economic comparison of road service.

There are eight major surface types (21), as defined by the Bureau of Public Roads, for which individual service-life analyses are made:

Soil-surfaced road. - A road of natural soil, the surface of which has been improved to provide more adequate traffic service by the addition of a course of mixed soil having A-1 or A-2 characteristics, such as sand-clay, soft shale or top-soil, or the addition of an admixture such as bituminous material, portland cement, calcium chloride, sodium chloride, or fine granular material (sand or similar material).

Gravel or stone road. - A road the surface of which consists of gravel, broken stone, slag, chert, caliche, iron ore, shale, chat, disintegrated rock or granite, or other similar fragmental material (coarser than sand) with or without sand-clay, bituminous, chemical, or portland-cement stabilizing admixture or light penetrations of oil or chemical to serve as a dust palliative.

Bituminous surface-treated road. - An earth road, a soil surfaced road, or a gravel or stone road to which has been added by any process a bituminous surface course, with or without a seal coat, the total compacted thickness of which is less than 1 inch. Seal coats include those known as chip seals, drag seals, plant-mix seals, and rock-asphalt seals.

Mixed bituminous road. - A road the surface course of which is 1 inch or more in compacted thickness composed of gravel, stone, or similar material, mixed with bituminous material under partial control as to grading and proportions.

Bituminous penetration road. - A road the surface course of which is 1 inch or more in compacted thickness composed of gravel, stone, sand, or similar material bound with bituminous material introduced by downward or upward penetration.

Bituminous concrete, sheet asphalt, or rock-asphalt road. - A road on which has been constructed a surface course 1 inch or more in compacted thickness consisting of bituminous concrete or sheet asphalt, prepared in accordance with precise specifications controlling gradation, proportions, and consistency of composition, or of rock asphalt. The surface course may consist of combinations of two or more layers, such as a bottom and a top course or a binder and a wearing course.

Portland-cement concrete road. - A road consisting of portland-cement concrete with or without a bituminous wearing surface less than 1 inch in compacted thickness.

Brick or block road. - A road consisting of paving brick, stone block, wood block, asphalt block, or other form of block, with or without a bituminous wearing surface of less than 1 inch in compacted thickness.

These eight major surface types have been combined into three groups, the low type, the intermediate type, and the high type. The first two surface types are low types, the following two surface types are intermediate types, and the last four surface types are high types. These combinations are arbitrary and general in nature.

These surface classifications are completely relevant to pit roads. The roads necessary to support heavy-duty trucks are thicker and stronger versions of the road types listed here. The high type road is often referred to as a heavy-duty road. This paper deals primarily with two different high type roads, the bituminous concrete road, and the portland-cement concrete road.

Roadway Service Life. The average service life of a roadway surface is the average period of time after construction that the surface remains in service prior to being replaced, resurfaced, reconstructed, or otherwise taken out of service.

In order to determine the advantages of constructing different kinds of roads it is necessary to be able to estimate the service life of these roads. The Bureau of Public Roads made a survey of state highways in 1941 in order to make an analysis of service-life characteristics of primary rural highway surfaces. Additional data were added and a new analysis was made in 1948. Finally, in 1956 data were compiled for a 50 year period and an analysis was made of the service-life characteristics of highways based on data submitted by 25 states (22).

As the number and weight of vehicles using roads has increased the service life of the different types of roads has decreased, even with improved road construction methods. The average service life of the different surface types over a 50 year period follows: soil surfaced - 5.2 years, gravel or stone - 8.3 years, bituminous surface treated - 12.6 years, mixed bituminous - 13.1 years, bituminous penetration - 18.0 years, bituminous concrete - 16.8 years, portland-cement concrete - 25.5 years, brick or block - 19.9 years. By 1952, when the latest data were compiled, these figures were reduced to about 2,3,9, 13,15,17, and 24 years, respectively, the brick or block types no longer being reported.

Recent data on the service life of state highways (23) indicate that the average duration before the first resurfacing of asphalt concrete is 18 years, and portland-cement concrete is 26 years.

Roadbed Structural Members. The BPR list of surface types shows the varied types of road construction which are possible. Most roads have additional structural components besides the surface. The surface courses on flexible pavements are constructed in order to protect the road from weathering, to provide a smooth riding surface, and to keep vehicles from destroying the main supporting roadbed. The bituminous concrete does not structurally support a superimposed load, but distributes the load to the supporting base and subbase. Bituminous concrete is a viscoelastic material. The surface course of portland-cement concrete pavement is an elastic structural member which directly supports the superimposed load and distributes this load to the subbase in a fairly uniform manner. The maximum subgrade stress is usually less than 5 - 10 pounds per square inch.

High-type flexible pavements are usually made up of a wearing-surface course, a base course of select-grade imported soil, and a subbase course which may consist of native soil or imported soil.

High-type rigid pavements consist of concrete slabs supported by subbase courses which may be native soil or an imported soil. Often the upper 4 - 6 inches of the subbase is either imported material or cement-treated native soil. This is often referred to as a base course.

Aggregate Base Material. The base and subbase structural materials which are used are classified in several ways. These materials may be classified by their Atterburg limits, by their field or laboratory bearing tests, or by their particle size grading limitations, or a combination of these methods (9,24). The bearing tests most commonly used are the CBR test, the triaxial tests, and the plate bearing test.

The sorted import material which is usually used to construct the base and subbase sections is often classified as Type "A", "B", or "C" material (25). The three types may not have a plasticity index greater than 6.0% or an expansion limit greater than 0.60%. The CBR of Type "A" is 80% - 100%, of Type "B" is 80% - 100%, and of Type "C" is 50% - 100%.

Type "A" base material shall be composed of a well blended combination of crushed rock and rock dust. The crushed rock shall be uniformly graded between 1 inch size and No. 8 screen size with the rock dust to be less than 12% of the total amount of material.

Type "B" base material shall be composed of a well blended combination of crushed rock, rock dust, sand, and gravel. The material shall be uniformly graded between 1 inch size and No. 50 screen size with the rock dust to be less than 10% of the total amount of material. That portion of Type "B" base material retained on the No. 8 sieve shall contain not less than 50%, by weight, of crushed rock.

Type "C" base material shall be the product of a rock crushing plant consisting of $1\frac{1}{2}$ inch size rock mixed with No. 8 screen size rock in the proportion of about 2 parts to 1. That portion of Type "C" base material retained on the No. 8 sieve shall contain not less than 50%, by weight, of crushed rock.

CHAPTER 4

ROADBED CONSTRUCTION COSTS

Cost Factors. There are no set costs for the construction of roadways in the United States. The type of roadway which is constructed is dependent upon the haulage service which is to be provided. The main determining factors in the design of roads are load, repetition, and soil bearing capacity. There are many other factors which must be considered also, such as terrain and climate (26). These main factors determine the type of roadway which must be built to sustain the kind of service required. When a private company wishes to provide a haulage road the cost factor often determines whether a road is to be built and what type of road will suffice.

Each roadway construction project is a unique problem with its own limiting factors. A general guide can be followed for cost estimating procedure but the special factors present will determine what kind of roadway can be constructed. Incremental cost studies of pavements (27) concerning tonnage versus cost have been undertaken but the results have been inconclusive due to the large number of variable factors which must be treated.

Comparative Pavement Analysis. In order to provide a guide for pavement cost estimation procedures some examples are provided which give a comparative construction cost analysis of bituminous concrete pavement and portland-cement concrete pavement. The unit prices listed are for materials supplied in-place in the vicinity of Los Angeles County during 1962. When determining materials costs for a construction project the cost of transportation and the availability of construction equipment will also have to be considered. Many adjoining pits may provide an economical supply of crushed rock which would reduce the pavement materials cost considerably. Table I contains a list of some materials unit prices and some assumptions concerning design criteria. Table II contains flexible pavement cost data and Table III contains comparable rigid pavement cost data.

Cost Index. The cost of materials and labor varies with time so that it is difficult to compare construction costs from year to year, and sometimes from month to month. Not only the materials and labor costs change but the relative value of money is in a constant state of flux. In order to relate these three variables so that relative costs can be compared from one time to another various cost indices are computed and maintained. Some states, such as California, maintain such indices. The most commonly used construction cost indices for roads are the one maintained by the Bureau of Public Roads (28) and the one maintained in the publication 'Engineering News-Record' (29).

The ENR construction cost index is very advantageous when comparing cost studies made on different projects in different years. The ENR index is usually listed for each study (30). To bring the cost figures to the date needed, divide the ENR index of that date by the index of the date when the study was made and multiply the costs by that factor.

Flexible Versus Rigid Pavements. Many studies have been made

TABLE I

DESIGN AND COST ASSUMPTIONS

In-Place Unit Costs

Item	Cost
Aggregate Base Type "A"	$6.00 per cu. yd.
Aggregate Base Type "B"	4.00 per cu. yd.
Excavation	1.25 per cu. yd.
Asphaltic Concrete Paving:	
2 inch thickness	0.10 per sq. ft.
4 inch thickness	0.16 per sq. ft.
6 inch thickness	0.25 per sq. ft.
Portland-Cement Concrete	25.00 per sq. yd.
Reinforcing Bars and Dowels	0.15 per lb.
Rough-Grading Compaction of Subgrade	0.03 per lin. ft.

Design Assumptions

ENR construction cost index is 880.

Modulus of rupture of concrete is 700 psi.

Allowable flexural stress of concrete is 350 psi.

Concrete slabs are 20 feet long and 18 feet wide of uniform cross-section with tongue-and-groove longitudinal junction.

CBR of base material is 80+, and CBR of subbase material is 50+.

A subgrade compaction of 90% of maximum density and a base and subbase compaction of 95% and 100%, respectively.

Excavation is equal to the depth of roadbed structure less three inches.

The flexible pavement base and the rigid pavement subbase consist of Type "A" aggregate. The flexible pavement subbase consists of Type "B" aggregate.

Two 7-foot wide aggregate shoulders are necessary to all roads. The shoulders shall have a structural section consisting of Type "A" aggregate equal in depth to the sum of the surface and base layers. The cost of the shoulders has not been included in the pavement cost analyses.

TABLE II

COMPARATIVE PAVEMENT ANALYSIS

FLEXIBLE PAVEMENT

			Design				In-Place Cost				
			Thickness in inches				Cost in dollars				
CBR	'k'	'd'	Surface	Base	Subbase	Excav.	Surface	Base	Subbase	Excav.	Total
			Case I	60,000 lb.	Single-Wheel Load						
2	50	58"	6"	16"	34"	53"	$9.00	10.65	15.50	7.35	$42.50
3	100	45	6	12	25	40	9.00	8.00	11.10	5.55	33.65
10	200	21	5	8	7	17	7.55	5.35	5.15	2.35	20.40
40	400	10	5	4	--	6	7.55	2.65	--	0.85	11.05
			Case II	40,000 lb.	Single-Wheel Load						
2	50	49"	5"	16"	27"	45"	$7.55	10.65	12.00	6.25	$36.45
3	100	38	5	12	20	34	7.55	8.00	8.90	4.75	29.20
10	200	18	4	8	5	14	5.75	5.35	2.20	1.95	15.25
40	400	9	4	4	--	5	5.75	2.65	--	0.70	9.10
			Case III	20,000 lb.	Single-Wheel Load						
2	50	36"	4"	16"	15"	32"	$5.75	10.65	6.65	4.45	$27.50
3	100	28	4	12	11	24	5.75	8.00	4.90	3.35	22.00
10	200	13	4	8	--	9	5.75	5.35	--	1.25	12.35
40	400	8	4	3	--	4	5.75	2.00	--	0.55	8.30

Note

Pavement width is 36 feet. (18 foot lanes).
Costs to nearest $0.05 per linear foot.
1" of surfacing is structurally equivalent to 1.3" of base.
'd' is the total structural thickness of pavement.

TABLE III

COMPARATIVE PAVEMENT ANALYSIS

RIGID PAVEMENT

		Design				In-Place Cost				
		Thickness in inches				Cost in dollars				
CBR	'k'	'd'	Cu. Yd. per Ft.	Sub-base	Ex-cav.	Con-crete	Sub-base	Ex-cav.	Dow-els	Total
		Case I	60,000 lb. Single-Wheel Load							
2	50	16.50"	1.83	6"	19.50"	$45.75	4.00	2.70	3.85	$56.30
3	100	15.75	1.75	6	18.75	43.75	4.00	2.60	3.85	54.20
10	200	14.75	1.64	4	15.75	41.00	2.65	2.20	3.85	49.70
40	400	13.50	1.50	4	14.50	37.50	2.65	2.00	3.85	46.00
		Case II	40,000 lb. Single-Wheel Load							
2	50	13.75"	1.53	6"	16.75"	$38.20	4.00	2.30	2.95	$47.40
3	100	13.00	1.44	6	16.00	36.00	4.00	2.20	2.95	45.15
10	200	12.25	1.36	4	13.25	34.00	2.65	1.85	2.95	41.45
40	400	11.25	1.25	4	12.25	31.20	2.65	1.70	2.95	38.50
		Case III	20,000 lb. Single-Wheel Load							
2	50	10.00"	1.11	6"	13.00"	$27.80	4.00	1.80	1.50	$35.10
3	100	9.50	1.06	6	12.50	26.50	4.00	1.70	1.50	33.70
10	200	9.00	1.00	4	10.00	25.00	2.65	1.40	1.50	30.55
40	400	8.25	0.92	4	9.25	23.00	2.65	1.30	1.50	28.45

Dowels: 16 inches in length spaced 12 inches c/c and transmitting 45% of the load.

60,000 lb. load - 36 dowels 2.00" diam. - $76.80/joint.
40,000 lb. load - 36 dowels 1.75 diam. - 58.85/joint.
20,000 lb. load - 36 dowels 1.25 diam. - 30.00/joint.

Note

Pavement width is 36 feet (18 feet per lane).
Transverse joints at 20 foot intervals.
Costs to nearest $0.05 per linear foot of pavement.
'd' is the concrete slab thickness.

in order to compare the advantages of rigid and flexible pavements (23,31,32). The flexible pavements are more economical to construct although the average rigid pavement has a longer service life. When the two types of pavements are compared on the basis of cost and service life, the determination of the type of pavement to use depends upon the type of soil present. The rigid pavement is economically advantageous for soils with a very low bearing capacity (CBR 2), the rigid and flexible pavements are of comparable cost for soils with a CBR of 3 - 5, and the flexible pavement is economically advantageous for soils with a higher bearing ratio.

Continuous Reinforcement. Concrete pavement with continuous reinforcement (33) has been used in the design of modern highways since about 1950. The amount of reinforcing steel necessary for very heavy design concrete slabs may be economically prohibitive. The longitudinal reinforcing steel should be equal to about 0.5% of the slab cross-sectional area and the transverse reinforcing steel should be greater than the concrete slab tensile stress (34,35,36). In considering the use of continuous reinforcement in the concrete slab, some possible design methods were investigated. If the listed design limitations are followed the cost of the steel reinforcement in-place would vary from a minimum of $12.75 to a maximum of $21.80 per linear foot of slab for thickness ranging from 8.25 inches to 16.50 inches. The reinforcing steel considered is one-half inch deformed bars.

There is a possibility that the heavy slabs may need less steel reinforcement. On this basis a cost estimate was made for one-half inch deformed bars 6 inches c/c longitudinally and 12 inches c/c transversely. This is considered insufficient reinforcement, yet the cost is still $10.80 per linear foot for the steel reinforcement. This is as much as one-quarter to one-half of the total construction cost of non-reinforced concrete slab roads.

Maintenance Costs. The maintenance costs of a roadway are determined by the same factors which dictate what kind of pavement is to be constructed. The maintenance costs vary with respect to the type of pavement and the age of that pavement. The Bureau of Public Roads compiles data on highway maintenance costs (37,38,39). These data are published by the Bureau of Public Roads and the Highway Research Board at regular intervals. A highway maintenance cost index is used to correlate the data with respect to time.

The two types of roads which are considered here are, again, the bituminous concrete (flexible) pavement and the portland-cement concrete (rigid) pavement. The maintenance costs for a pit road will be similar to the maintenance costs for very heavy design interstate highways. The maintenance costs may include sweeping costs. The Asphalt Institute (31) and the Stanford Research Institute (23) have compiled comparative data on rigid and flexible pavement highway maintenance costs. The average maintenance cost over a 26 year period is about $250 per mile-year. The initial flexible-pavement maintenance cost is about $50 per mile-year, rising to about $300 per mile-year after 18 years, at which time a new surface layer is added and the maintenance cost is lowered to about $250 per mile-year. The initial rigid-pavement maintenance cost is about $80 per mile-year, rising to about $150 per mile-year by the end of the fifth year, then rising to about

$250 per mile-year after 26 years. If the required load capacity and volume of traffic remain unchanged by the end of the 26 year period, the rigid pavement can be surfaced with asphalt and kept in service for a number of years with no greatly increased maintenance cost.

CHAPTER 5

TRUCK PERFORMANCE

Factors Affecting Performance. The three main aspects of truck performance which are most important when considering road construction are rolling resistance, traction, and tire service. There are other problems to be considered but these three main aspects will determine the truck service life and the haulage costs for any type of road which may be used.

Two other road conditions which affect truck performance are roughness and dust. When low type roads are used as permanent haulage roads these factors become economically important. These conditions are corrected by use of repetitive grading and distribution of dust palliatives. These dust palliatives may consist of water or such chemicals as lignin sulfonate (40). The costs of controlling road roughness and dust may be so great that a more durable road would prove feasible. Grading costs are about $10.00 per mile (38, 39) and may need to be repeated more than once per day. Dust palliatives are also expensive. Water treated surfaces are often inadequate and lignin sulfonate treatment costs as much as $1500 per mile. This treatment ordinarily only lasts a few weeks. The lignin sulfonate treatment would not work on plastic soils since the truck tires would ravel the treated surface exposing the dust beneath.

Rolling Resistance. The rolling resistance of a truck is the retarding force of the ground against the wheels of the vehicle. This resisting force has been measured by many vehicle manufacturers. Table IV is a list of average values which have been determined for trucks (41).

TABLE IV

APPROXIMATE ROLLING RESISTANCE

Type of Surface	High Pressure Tires		Low Pressure Tires	
	Lbs. per Tn. Gross Wt.	Per Cent Gross Wt.	Lbs. per Tn. Gross Wt.	Per Cent Gross Wt.
Smooth Concrete	30	1.5%	40	2.0%
Smooth Asphalt	36	1.8%	48	2.4%
Smooth Hard Gravel	45	2.3%	60	3.0%
Smooth Hard Dirt	60	2.5%	80	4.0%
Earth Roads Dry Dusty	110	5.5%	90	4.5%
Unplowed Earth	150	7.5%	100	5.0%
Plowed Earth	190	9.5%	160	8.0%
Earth Road Rutted	210	10.5%	180	9.0%
Loose Sand and Gravel	280	14.0%	240	12.0%
Deeply Rutted or Plastic Base	350	17.5%	320	16.0%
Snow Packed	50	2.5%	70	3.5%

The rolling resistance is often defined to include the bearing friction of the wheels and the tire flexing. These resisting effects are small in comparison to the resistance caused by tire penetration, especially on non-cohesive or plastic soils.

This investigation primarily concerns high-type roads. All road surfaces deflect when loaded so that even on a level constructed surface such as asphalt or concrete pavement a vehicle is always running up a slight grade caused by the load induced deflection.

The rolling resistance of a haulage road increases haulage costs in several ways. The greater the tire penetration, the greater the rolling resistance. Added power is required to overcome this resistance. This added power is expensive as shown by Table IV. The power used to overcome rolling resistance leaves less power for maintaining haulage speed. A lower required speed means using a lower gear speed with an ensuing greater cost. This lower gear speed means a longer haulage time will be needed, and the added time not only directly causes a haulage cost increase but also indirectly since, to maintain the pit tonnage output, additional trucks must be used.

Traction Loss. Another reason for considering the construction of high-type pit haulage roads is the loss of time and energy caused by traction loss. Table V is a list of the approximate coefficients of traction for the different types of road surfaces (41).

It is evident that some road surfaces may cause high haulage costs due to loss in power and time. This table also introduces a good argument for keeping a high-type road surface cleaned.

Tire Service. The tires manufactured for use on heavy-duty

TABLE V

APPROXIMATE COEFFICIENTS OF TRACTION

Materials	Rubber Tires
Concrete	.90
Dry Clay Loam	.50 - .58
Wet Clay Loam	.40 - .49
Loose Sand	.20 - .35
Rock Quarry	.60 - .70
Gravel Road	.36
Packed Snow	.20
Ice	.12
Firm Earth	.50 - .60
Loose Earth	.40 - .50

off-highway trucks are different than ordinary truck tires used in highway haulage service. In order to support a heavier load, the tires have thick walls with special reinforcing consisting of both organic and metallic fibers. These tires also have very thick and durable wearing surfaces. This very thickness restricts the service performance of the tire. The tires may not be run at high speeds since the flexure of the walls of the tire would generate sufficient heat to break down the rubber.

Tire life estimates range from 10,000 miles to 60,000 miles depending on job conditions. The conditions upon which tire life depend include such items as maintenance, speed, surface conditions, wheel positions, loading, grades, and tire inflation (41). Table VI is a list of tire life expectancy data for wheel-type tractors and hauling units (42).

Tires for pit haulage trucks consist of two main types, the

TABLE VI

TIRE LIFE EXPECTANCY TABLES

	Excellent Conditions	Average Conditions	Severe Conditions
Estimated Mileage	60,000 miles	45,000 miles	24,000 miles
Average Speed	12 mph	10 mph	8 mph
Average Life	5,000 hours	4,500 hours	3,000 hours

wide-base tires and the regular truck tires. The wide-base tires are used by vehicles such as the earth movers. These tires have speed restrictions. One line of tires is restricted to haulage speeds under 10 mph and the other line of tires is restricted to haulage speeds under 30 mph. Both the wide-base and the regular 10 mph tires will haul about 10% more than the 30 mph tires for the same tire cost. Figure 1 shows the approximate fleet price tire cost per maximum tire load rating. In general the 10 mph tires tend toward the minimum cost curve and the 30 mph tires tend toward the maximum cost per load. The fleet price is approximately two-thirds of the list price. The tire cost data is taken from the average tire prices of the major tire companies. Only the standard tire sizes were investigated.

In order to determine the size and type of tires which are available for pit haulage, a copy of the standards issued by the Tire and Rim Association (43) was used. Figures 2, 3, 4, and 5 are compiled from tables listed by the Tire and Rim Association. These data are recommended standards for these tires. Wherever an 'x' occurs on these plotted

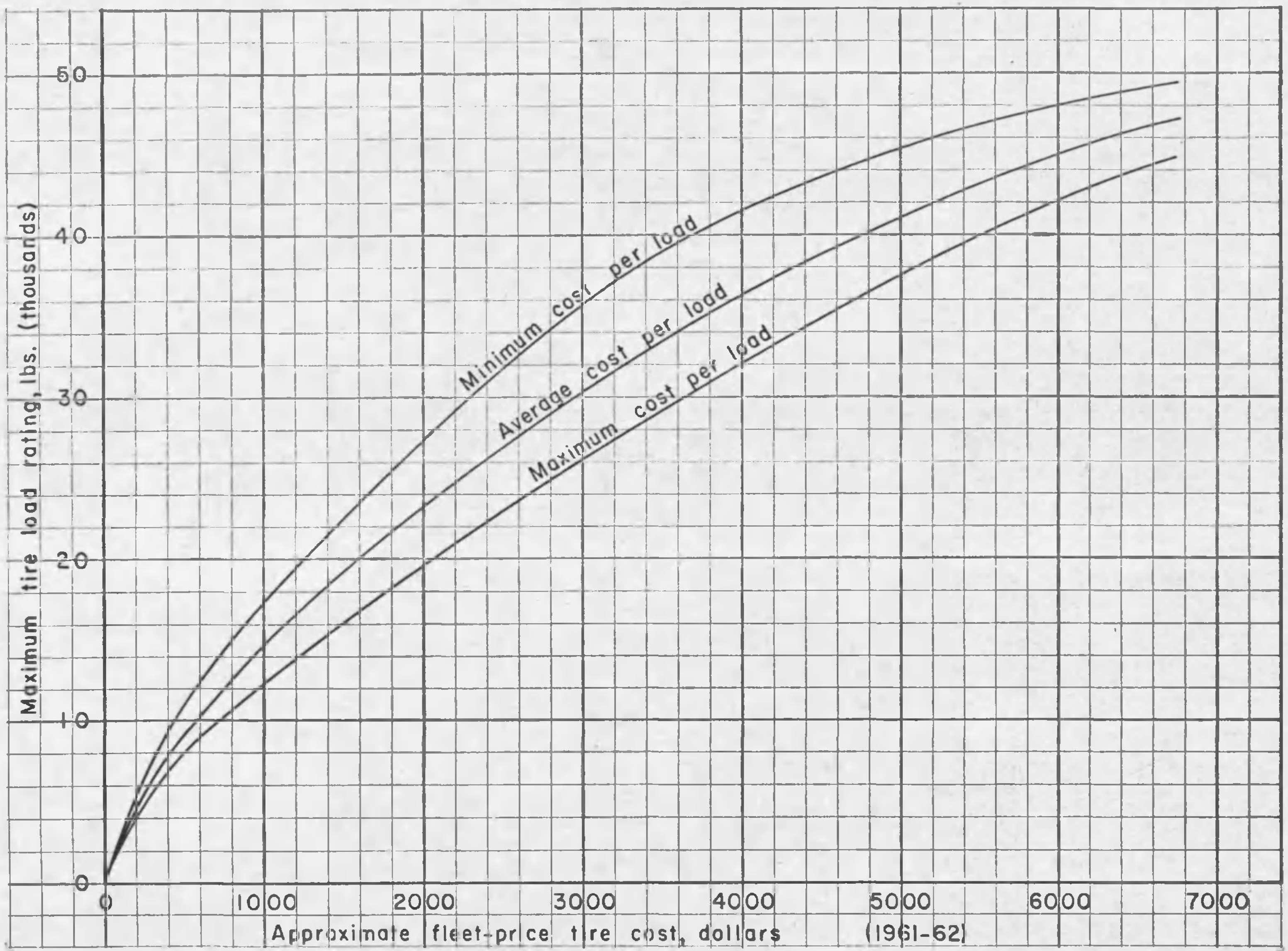

Figure 1. Tire prices for heavy-duty off-highway trucks.

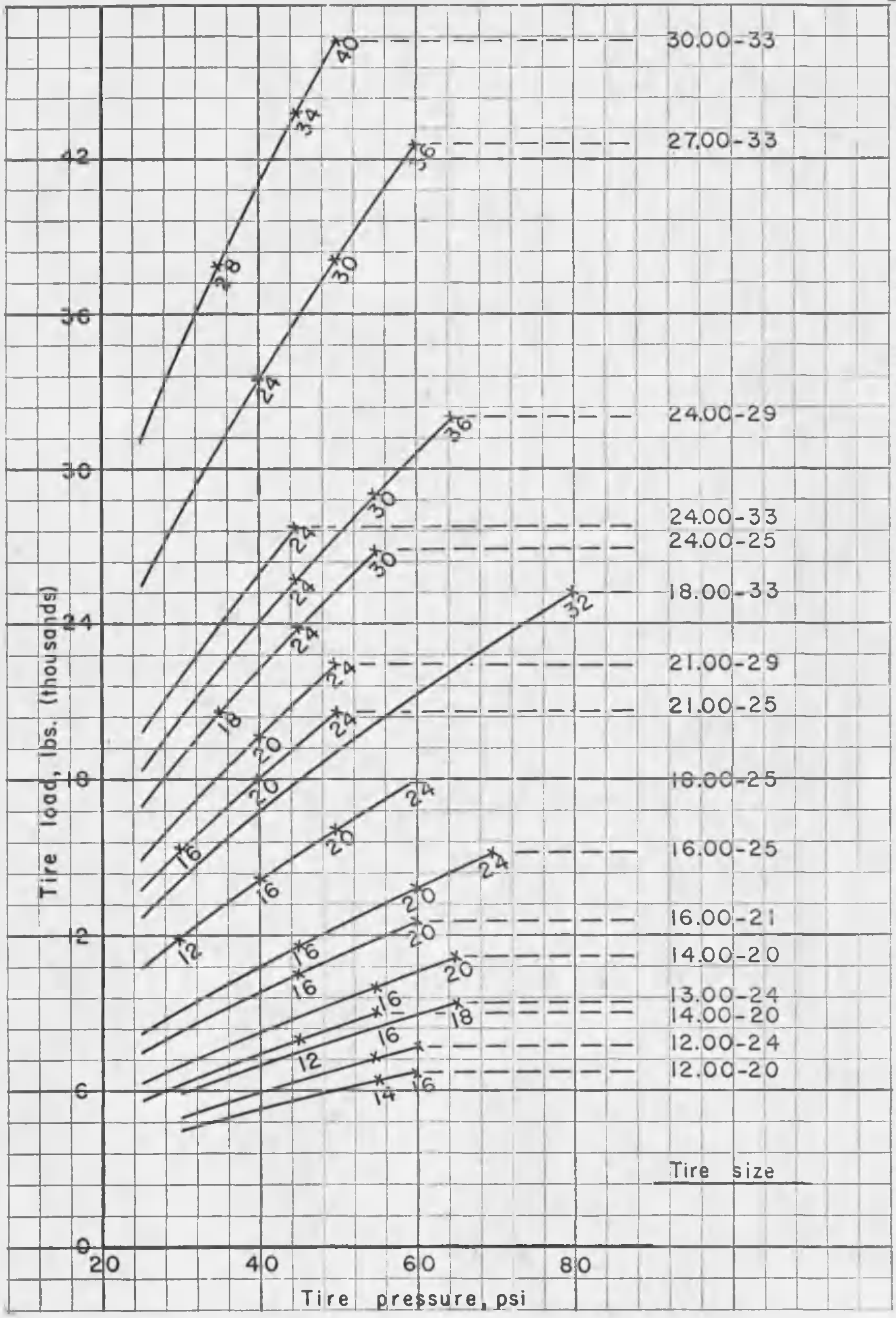

Figure 2. Tires for earth-moving and mining service for short hauls. Maximum speed - 30 mph.

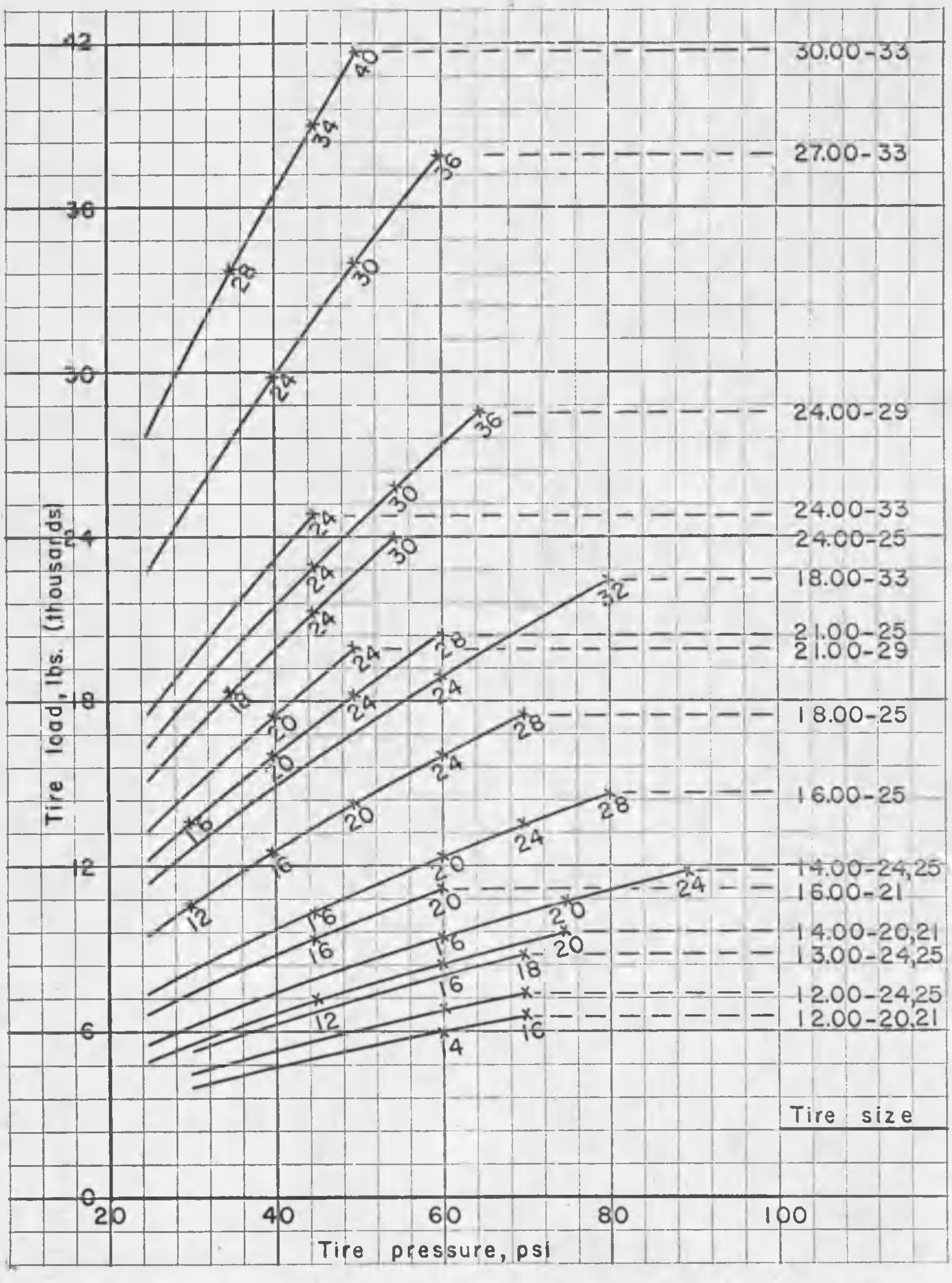

Figure 3. Tires for earth-moving service for short hauls. Maximum speed - 10 mph.

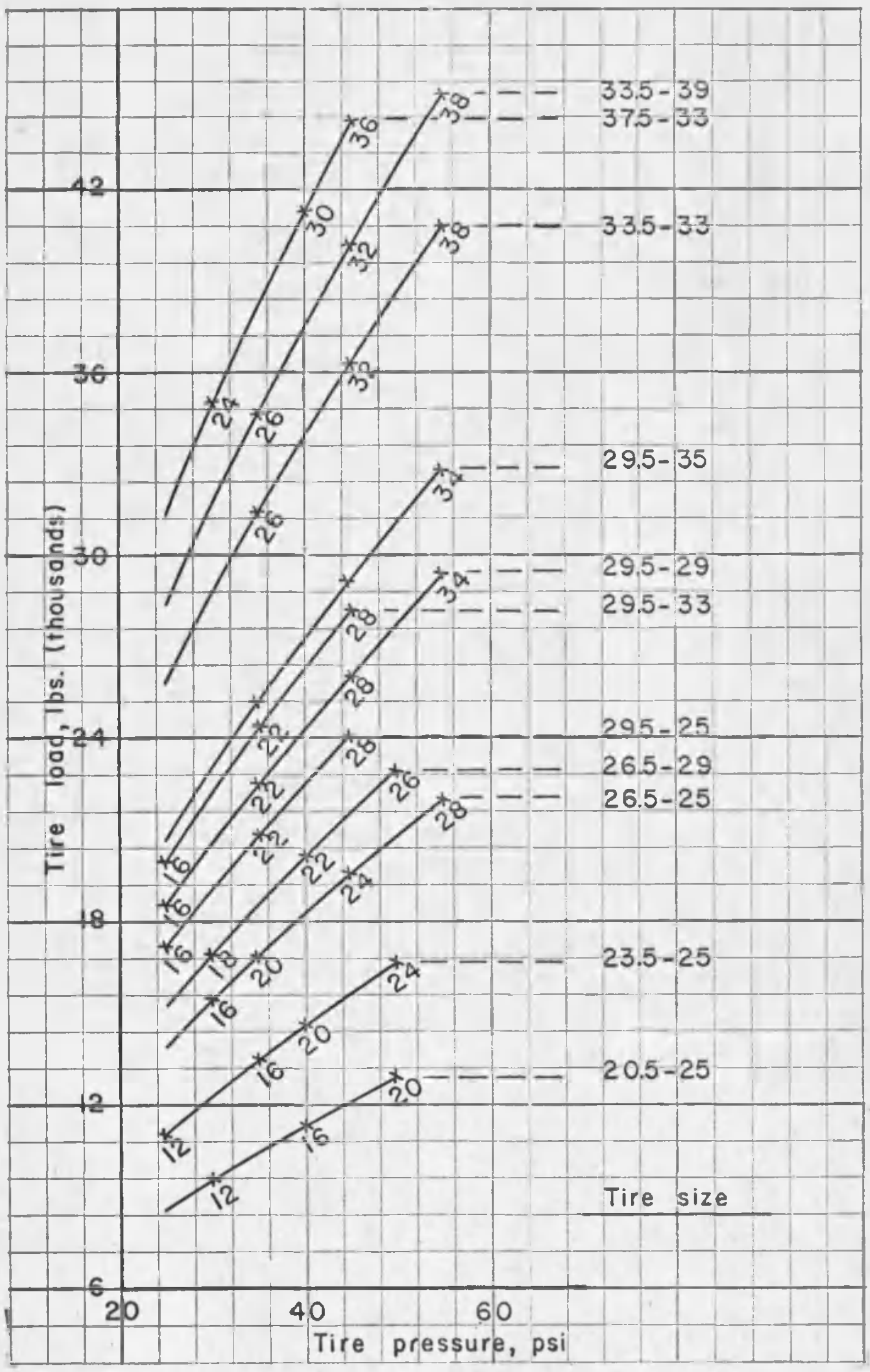

Figure 4. Wide-base tires for earth-moving service for short hauls. Maximum speed - 30 mph.

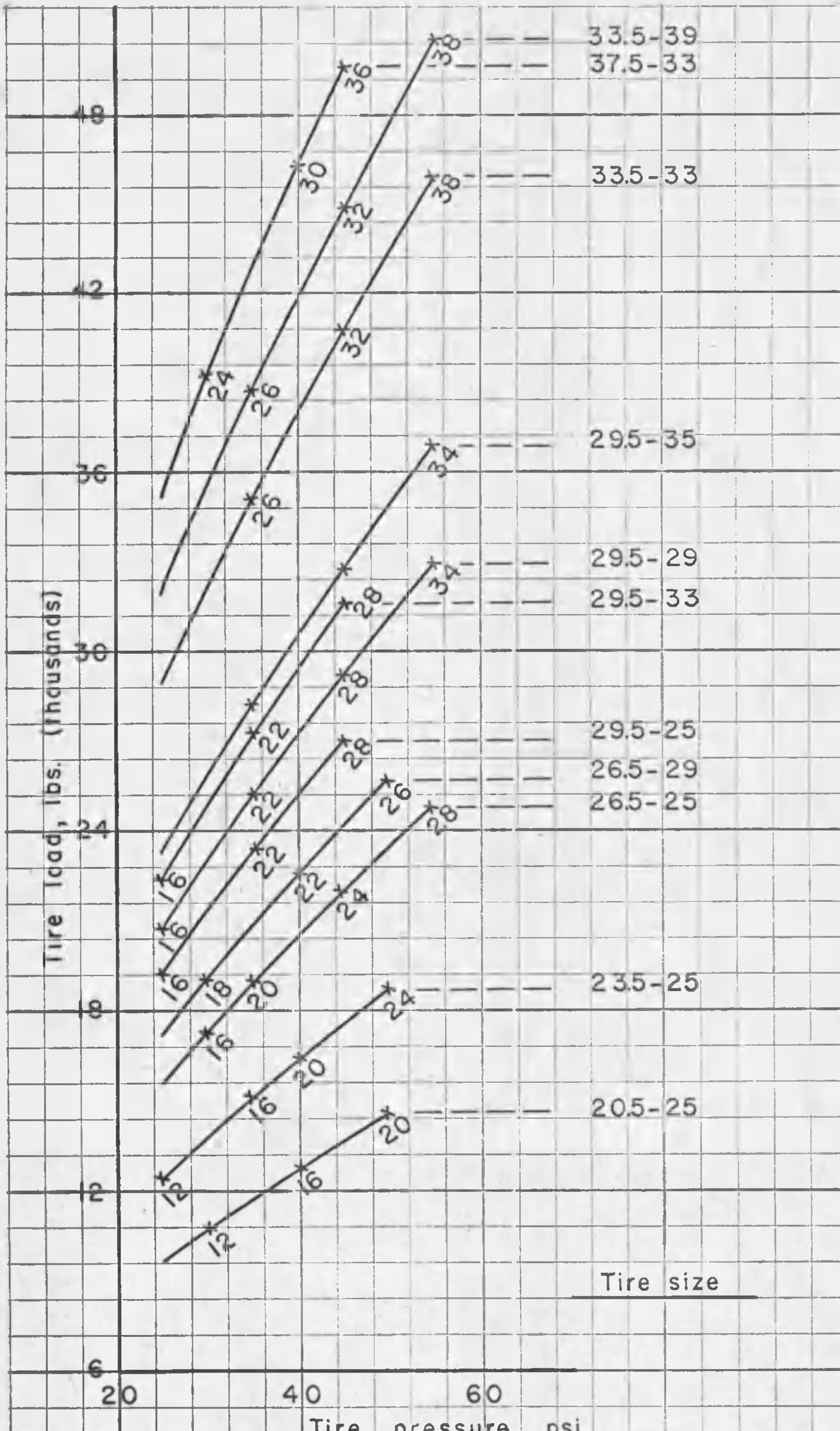

Figure 5. Wide-base tires for earth-moving service for short hauls. Maximum speed - 10 mph.

curves the maximum recommended load for the ply rating shown below the 'x' is denoted. With the aid of these four tire charts it is possible to determine tire costs for the different tire sizes. Compare the tire maximum-load capacities with the price estimations given in Figure 1.

Tires are fabricated in such a way as to follow definite rules. The rather narrow limits of the two envelope curves in Figure 1 indicate this. In order to determine the manner in which tires are fabricated, the maximum-load tire inflation pressure is plotted against the maximum tire load rating in Figure 6. The curves indicate the average ply rating. The solid curves indicate the tires which are ordinarily manufactured and the dashed lines are extrapolated values.

The tendency is to manufacture tires with larger load ratings since such arrangements as tandem axles give a much shorter tire life. These larger tires, as shown in Figure 6, tend to have lower tire pressures and larger ply ratings.

The tire pressure factor becomes important when designing roads since the superimposed load is transmitted to the road through the contact area of the tires. This contact area is determined by the load, the tire inflation pressure, the tire size, the tire tread, and the type of pavement surface. Tire treads give contact areas ranging from about 45% of the total print (44) to 100% of the print. The transmitted tire load is usually considered as though there was a 100% contact area with a perfectly elastic tire. The contact unit pressure is usually considered equal to the tire load divided by the tire pressure (45,46). The tire contact area is usually considered as circular when

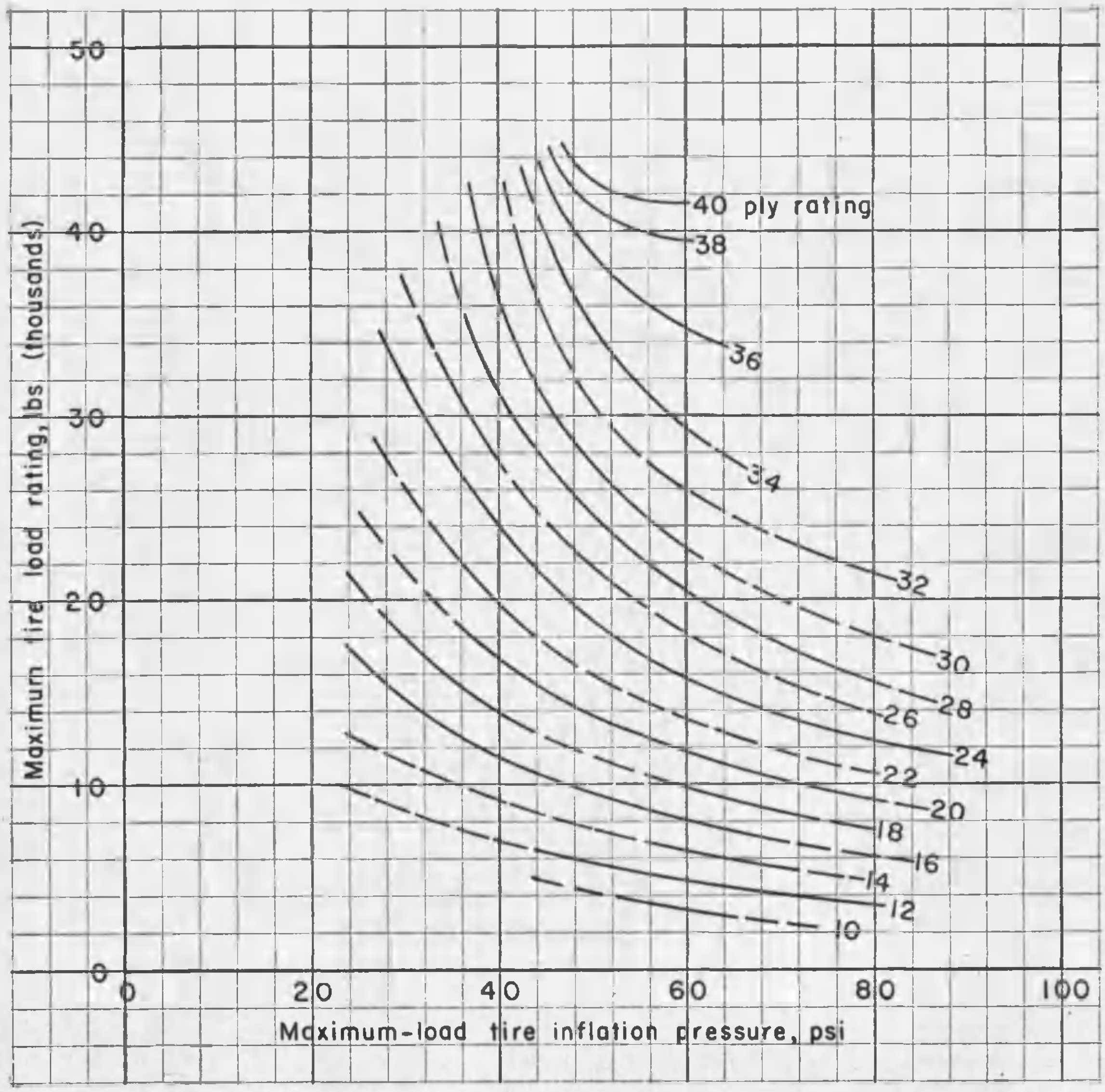

Figure 6. Load carrying capacity of heavy duty tires. Dashed lines indicate extrapolated values.

computing tire contact areas.

The contact area of tires is usually expressed as a radius of contact area (47). When computing the contact area for dual truck tires, the area between the dual tires is considered as part of the contact area. Since truck dual tires are set very close together the error induced by this method is small. Table VII is a list of standard rims and dual wheel spacings (43). This table is used in pavement design procedure.

The tire data from Figure 6 is used, as shown in Figure 7, in order to compute the radii of contact areas for the different standard tires at maximum loads. The maximum-load ratings were taken from Figure 6 at each five pound increment starting at 25 psi. These readings were taken for each ply rating. The maximum-load value at each point was divided by the tire pressure at that point. This value is the theoretical contact area of the tire. The radii of these contact areas were then computed. Each maximum-load tire rating is plotted against the comparable radius of contact area in Figure 7. A line indicating the average radius per load is shown. This chart is an extension of the one published by the Portland Cement Association (47). Since their data are based on highway truck service it is inadequate for very heavy pavement design. The data from Figure 7 can be used in roadway design. The average truck will not ordinarily be operated with the tires carrying their maximum rated load so the data from Figure 7 will have an inherent safety factor.

TABLE VII

TIRE RIMS AND DUAL-WHEEL SPACING

Tire Size	Preferred Rims	Minimum Dual Spacing	Tire Size	Preferred Rims	Minimum Dual Spacing
7.00-20	5.50	10.0 in.	12.00-21	8.50	15.0 in.
			12.00-25		
7.50-20	6.00	10.7			
			13.00-25	8.50	16.0
8.25-20	6.50	11.6		10.00	16.7
9.00-20	7.00	12.6	14.00-21	10.00	17.7
			14.00-25		
10.00-20	7.50	13.4			
10.00-22			16.00-21	11.25	20.2
10.00-24			16.00-25		
11.00-20	8.00	14.1	18.00-25	13.00	23.1
11.00-22			18.00-33		
11.00-24					
			21.00-25	15.00	--
12.00-20	8.50	15.0	21.00-29		
			24.00-25	17.00	--
13.00-24	9.00	16.2	24.00-29		
			24.00-33		
14.00-20	10.00	17.7			
14.00-24			27.00-33	22.00	--
			30.00-33		
		Tires Using Wide-Base Rims			
20.5-25	17.00	24.0	33.5-33	28.00	--
			33.5-39		
23.5-25	19.50	27.3			
			37.5-32	32.00	--
26.5-25	22.00	30.6			
29.5-25	25.00	--			
29.5-29					
29.5-33					
29.5-35					

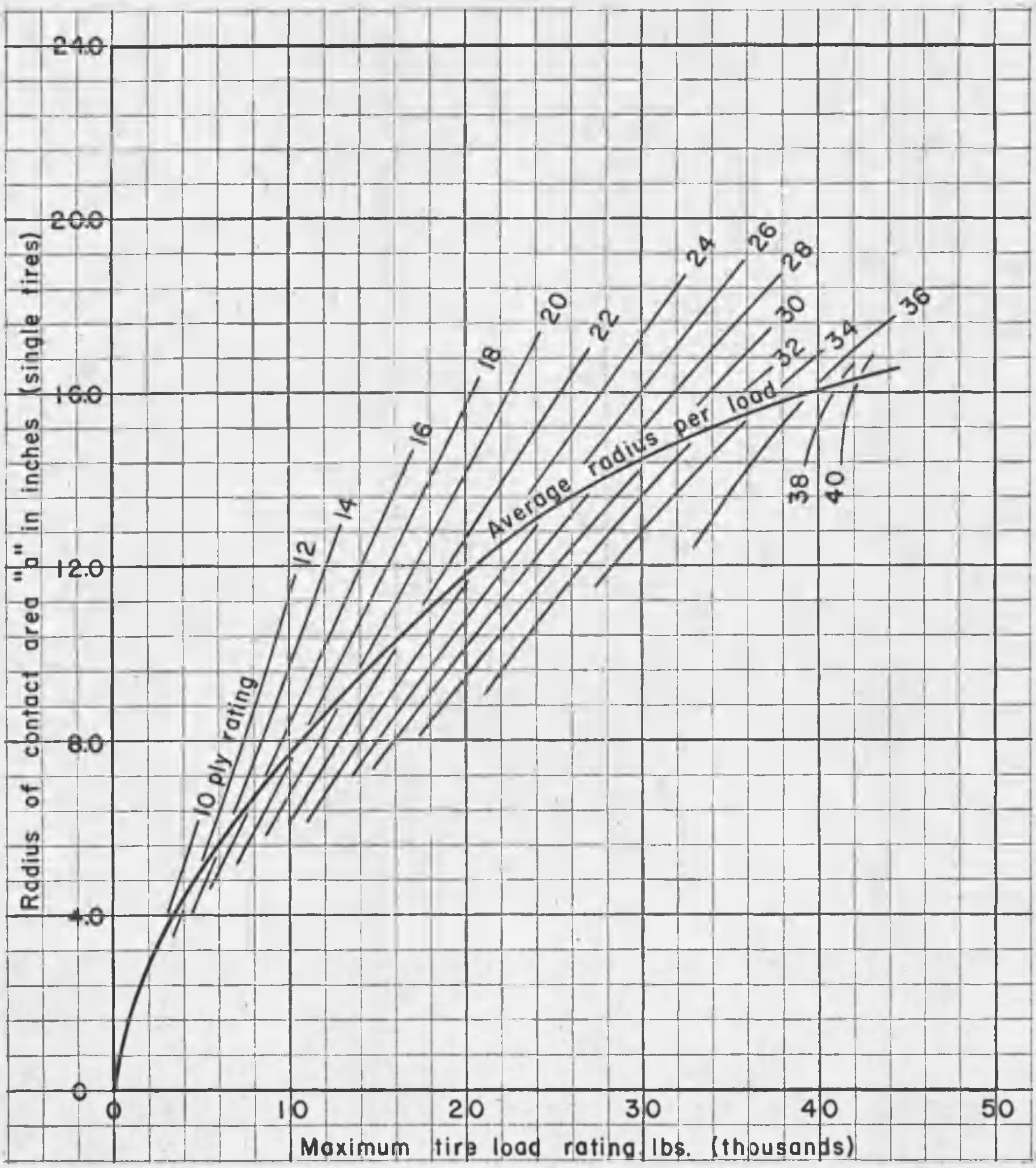

Figure 7. Radius of contact area for various wheel loads.

CHAPTER 6

FLEXIBLE PAVEMENT DESIGN

Design Factors. The structural design methods in use today are empirical or at least contain some factors which are of empirical origin (10). The flexible pavements in use before 1940 were not designed for wheel loads much greater than 10,000 pounds. There were exceptions to this limit but the roads designed for heavier loads were considered to be very special cases.

The heavier airplanes which were constructed during the second world war were too heavy to use the existing airport pavements. At that time it became necessary to prepare a design procedure for the construction of flexible pavement which would support heavy wheel loads (11,12). This design procedure is based upon the testing to destruction of specially built pavements of varying types of construction under repetitions of varying wheel loads.

It is now not uncommon to design pavements which will support airplanes weighing as much as 320,000 pounds. The design procedure for pavements which will support heavy-duty off-highway trucks is very similar to airport pavement design procedure (48). Pavement design procedures for heavy loads are an extension of the regular highway design methods.

Any pavement design method which is comprehensive must include at least three factors. It must have a factor to represent the destructive effect of traffic, a factor to evaluate the resistance to displacement of the soil under the protective layers of pavement and base, and a factor to evaluate cohesion or tensile strength of various layers in the structural section. The destructive effect of traffic includes both load and repetition.

The original flexible pavement design chart as proposed by Porter (11) was for flexible pavements which were constructed with untreated gravel base and subbase material. There was no cement, asphalt, or pozzolan mixed with the base material. This design chart has been tested and used by the Corps of Engineers (12). The method was adopted and is now in use as an official design procedure. Very few changes were necessary in this original design chart. The modern heavy aircraft use high pressure tires, some as high as 175 psi. This entails an increase in pavement thickness and a more durable surface (49). Since the heavy-duty off-highway trucks do not use high pressure tires this chart modification is unnecessary.

Figure 8 is a copy of the original proposed flexible pavement design chart. The asphaltic surface is ordinarily considered as part of the base material and included in the total roadbed thickness. The upper 6 - 8 inch layer of the subgrade beneath the roadbed is considered to be compacted to 80% of maximum density. The subgrade soil classification indicated in Figure 8 is general in nature and not sharply delineated. The minimum limits of roadbed thickness which were proposed are now used by the Corps of Engineers.

In order to prepare the design chart so that equivalent single-wheel loads may be computed, the design chart of Figure 8 was replotted

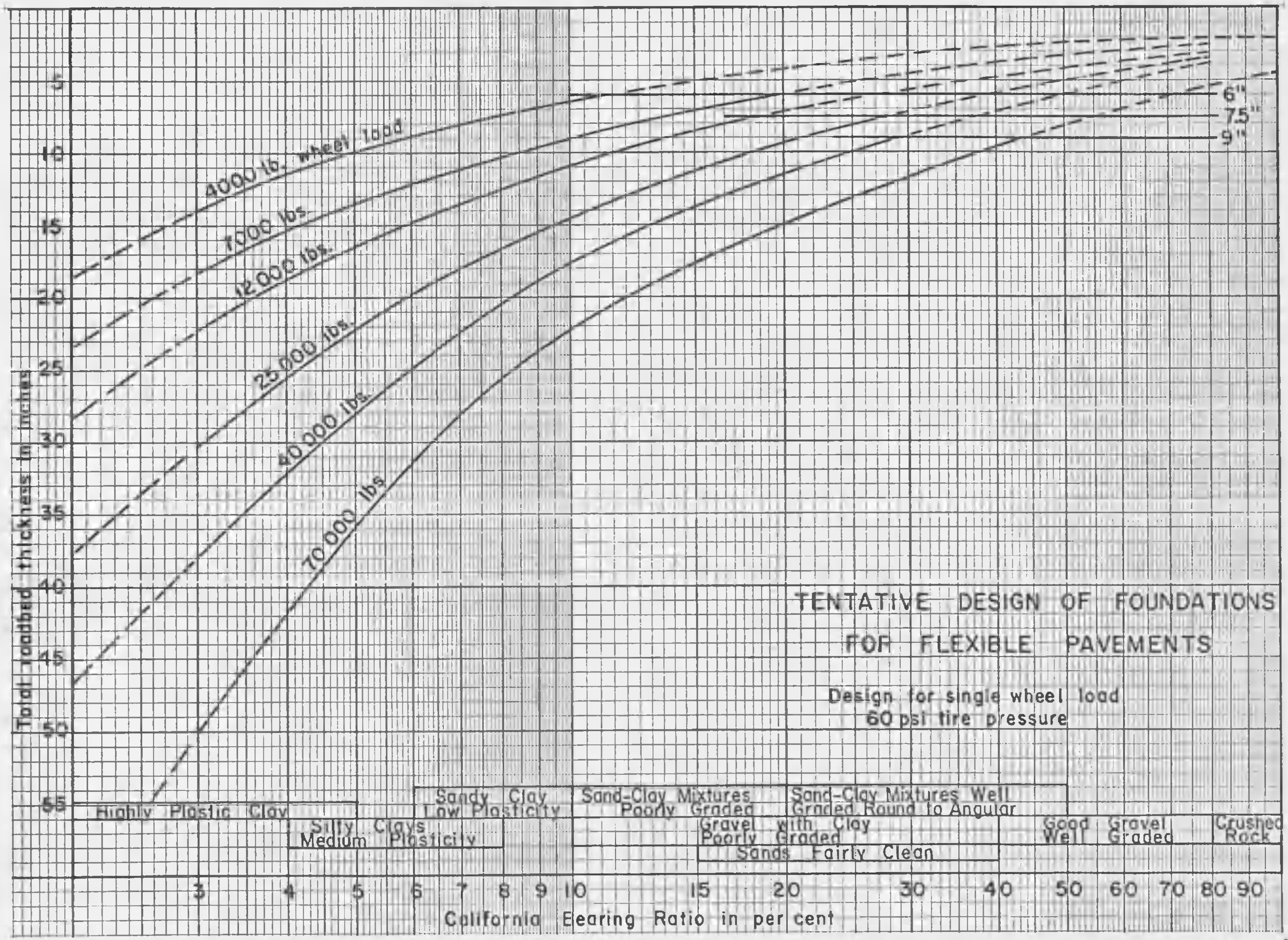

Figure 8. Original flexible pavement design curves.

on log-log graph paper (Figure 9). Either chart may be used to determine the total required roadbed thickness. It should be noted from the chart of Figure 9 that the CBR values have a rather precise logarithmic relationship. This relationship is so precise that the logarithmic scale on a graph paper can be used to interpolate further CBR values.

Equivalent Single-Wheel Load. In order to proceed with the structural design of a roadbed the maximum equivalent single-wheel load must be determined. This determination is based upon two different hypotheses. One hypothesis is that equal pavement deflections caused by two different loadings are equally destructive to the pavement (50,51). The other hypothesis is that equal pavement stresses caused by two different loadings are equally destructive to the pavement (52,53). If the materials in the roadbed were perfectly elastic these two hypotheses would give equal results. Even though the soil is not perfectly elastic the different methods give very similar results (50).

The CAA (Civil Aeronautics Administration) method of determining the equivalent single-wheel load will be used (53). The CBR soil classification is used instead of the FAA soil classification since it is more commonly used and a more precise classification. Use the chart on Figure 9 to carry out these computations. The following example shows the equivalent single-wheel load for a dual-wheel assembly. 'S' is the distance between the center lines of the two tire tracks and 'd' is the open distance between the two tire tracks. Table VII is a list of standard dual-tire spacings and rim sizes.

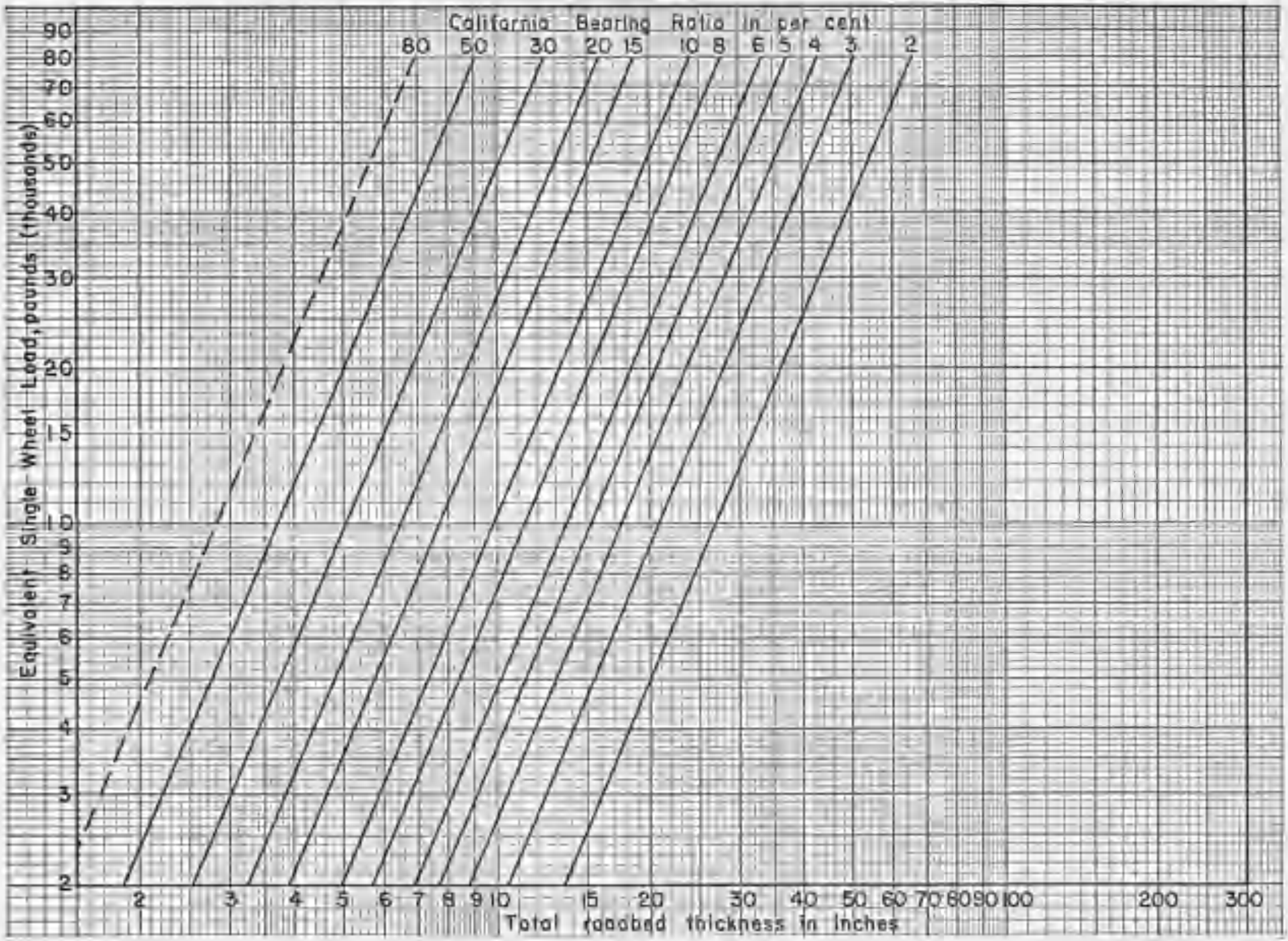

Figure 9. Design curves for flexible pavements.

Example:

Given: Dual wheel assembly load of 40,000 pounds, S = 23.1 inches, rim size = 13.00 inches giving a value of d = 10.1 inches, and the CBR of the subgrade is 10.

Find: d/2 = 5.1 inches is the depth of pavement to which each wheel acts as an individual single-wheel load of 20,000 pounds. Plot point on Figure 9.

2S = 46.2 inches is the depth of pavement at which the assembly acts as a single-wheel load of 40,000 pounds. Plot point on Figure 9.

Draw a straight line between these two plotted points. The intersection of this line with the CBR 10 line gives an equivalent single-wheel load of 29,000 pounds.

The tandem axle spacing and the track width of the axles are considered to be large enough so that there is no interference between the separate wheel loadings.

Load Repetition. Most flexible pavement design criteria include the destructive effect of traffic. The behavior of most materials under repeated loads is extremely complex (54). The loading and repetition factors are usually lumped together so that they cannot be separated. Many flexible pavement design methods have no place for a repetition factor, or the pavements are simply constructed with a large safety factor to cover possible repetitive load damage.

Research has been conducted on the problem of load repetition by the Highway Department of the State of California (10, 26). Recently the WASHO tests (5) and the AASHO tests (6) have added much to knowledge of the subject. It has been determined (10) that for a subgrade soil with a CBR of 4 to 5 the pavement thickness will vary so that

$T = K_1 W^{0.57}$ and $T = K_2 r^{0.113}$. T is the pavement thickness, r is the number of repetitions, and W is the wheel load. These factors can be combined so that

$$\Delta T = K_3 W^{0.57} \Delta r^{0.113} .$$

The value of K_3 is approximately 0.45, so that

$$\Delta T = 0.45\, W^{0.57} \Delta r^{0.113} .$$

This equation indicates that the effect of repetition also depends upon the load. A 40 kip load will give a ΔT of 5.7 inches in going from 10 to 110 repetitions, 7.5 inches from 100 to 1100 repetitions, and 10.0 inches from 1,000 to 11,000 repetitions. A 6 kip load will give a ΔT of 2.1 inches going from 10 to 110 repetitions, 2.7 inches going from 100 to 1100 repetitions, and 3.7 inches going from 1,000 to 11,000 repetitions.

The preliminary design curves proposed by Porter (11) had the repetition factor included in them so that it is apparent that as the CBR value of the subgrade increases the total effect of load repetition is considerably less. Until the effect of repetition of load has been determined for different CBR values, it is difficult to assign any separate values to this factor. About one-quarter of the design-load factor seems to be attributable to repetition of the load.

Another phase of the study of load repetition is the tandem effect. This is the relative destructive effect of tandem axle loadings as compared to an equivalent single axle load (5,10). One tandem-wheel load is considered equal to one single-wheel load 20% heavier than the tandem-wheel load. The tandem axle spacing on a heavy-duty truck may be as large as 7 to 8 feet instead of the average 4 foot axle spacing on a commercial truck. Without additional research it is not possible to determine the tandem effect for such large trucks. Consider the tandem axles as two separate axles when designing the pavement structural section.

Gravel Equivalents. The gravel equivalent is a thickness of gravel which has the same load carrying capacity as the section under consideration. The gravel equivalent is determined by the cohesive value of the material considered. The most common method of determining cohesive values is by means of the Hveem Cohesiometer (14, 26, 55, 56). The test results of this machine are stated in terms of cohesiometer values. This is a quantitative measure of the resistance to failure in tension of the test specimen.

The roadbed thickness required is inversely proportional to the 5th root of the cohesion value (10). Some cohesiometer values are listed in Table VIII. The equivalent inches of gravel for each type of material are shown.

The cohesiometer value is the cohesive strength of the asphalt concrete surface, and the cohesive strength of treated base and subbase material. Standard size samples are tested under uniform conditions and the results are expressed as grams/inch width, corrected to 3 inches height.

In order to determine the gravel equivalent the cohesiometer value of the material is divided by the cohesiometer value of gravel and the fifth root of the result is derived. This value, multiplied

TABLE VIII

COHESIOMETER VALUES

Type of Material	Cohesion Value	Equivalent Inches of Gravel
Type 'A' Cement Treated Base	1500	1.72
Type 'B' Cement Treated Base	750	1.50
Plant Mixed Surfacing (Paving Grade Asphalt)	400	1.32
Road Mixed Surfacing (Liquid Asphalt)	150	1.08
Untreated Base	100	1.00

by the thickness of treated material, gives the gravel equivalent thickness (57).

Now that the results of the AASHO road test are being correlated there is a growing disagreement concerning the gravel equivalent values. The above values are considered too conservative. This tends to give a large safety factor to this portion of the design procedure.

The amount of cement which is used in the cement treated base must be kept small, not only because of economy but because of the contraction cracking caused by the treated material. These cracks will tend to appear in the pavement surface. They are referred to as reflection cracks since they reflect the cracks below. Type 'A' base material should not have more than 3.5 - 6.0% cement, Type 'B' base material not more than 2.5 - 4.5%, and Type 'C' base material not more than 2.0 - 3.0%.

Sample Design Computations. Consider a load of 40,000 pounds on dual 18.00-33 tires with a ply rating of 32, a subgrade with a CBR of 10, and over 100,000 repetitions of stress. The equivalent single-wheel load was computed to be 29,000 pounds.

It is desired to haul at 30 mph rather than acquire larger, slower trucks. Figure 2 shows that the proposed tires are adequate to carry the load. It is necessary to introduce an impact factor. Ordinarily an additional 20% of the weight of the equivalent single-wheel load is added to the design load. 29,000 pounds x 1.20 gives a design load of 34.8 kips or 35,000 pounds.

Figure 9 gives a value of 17 inches of total roadbed thickness required to support 35,000 pounds with a subgrade CBR of 10. This 17 inches of structure is the structural section required if there are no special factors to be considered, such as frost, swelling, vibration, large plasticity index, or excessive repetition.

Consider the factors which may require an increase in structural section (Los Angeles County Road Department Design Standards).

Add 2 inches when the swell equals 5.0% to 6.5%, inclusive.
Add 2 inches when the elasticity equals 1.8% to 2.5%, inclusive.
If the swell or elasticity exceed these maximum percentages, excavation or soil stabilization is necessary.
Add 4 inches if the CBR is less than 3.

The base material should be at 100% of maximum density, the subbase should be at 95% density, and the upper 6 - 8 inches of the subgrade should be at 80% density (58,59,60). Generally, the structural section will consist of about one-third base material and two-thirds subbase material. The structure should have 4 - 5 inches of asphaltic plant mix surface, consider a 4 inch surface.

The required 17 inch structure less the 4 inch surface leaves 13 inches of required base and subbase. This 13 inches of structure may be made up of any proportion of base according to the thickness design chart. The type of material used may be economically determined (61). The above suggested proportion should be used when possible. The structural support afforded by different types of materials will vary (50, 62). Additional research is needed to make an adequate comparison between different base course materials. Interlocking of particles increases the bearing capacity of the base materials. Crushed slag or crushed limestone gives better support than crushed gravel. The base material should have a CBR of 80+ and the subbase material should have a CBR of 50+.

Consider 8 inches of Type 'B' subbase with a CBR of 50. The chart on Figure 9 shows that with a wheel load of 35 kips and a CBR of 50 a minimum 6.4 inch structural section is required on the 8 inch subbase layer. A 5 inch layer of Type 'A' base is adequate for the structure.

A substitute may be made for the 8 inches of Type 'B' subbase with a CBR of 50. On Figure 9 read 8 inches on the abscissa, then follow the 8 inch line vertically until intersecting the CBR 50 line, proceed horizontally across to the CBR 20 line, proceed vertically back down to the abscissa, and read 14 inches. The 14 inches of CBR 20 material may be substituted for the 8 inches of CBR 50 material. The compaction must still be to 95% of maximum density. If the CBR 20 material is readily available and the CBR 50 material is not, then the

14 inches of subbase is economically more advantageous than the 8 inches of Type 'B' subbase.

Treated Base. Treated base and/or subbase material may be used to construct a thinner structural section when it is economically advantageous (63). For example, if it is found that a native basement soil requires 17 inches of gravel cover for a satisfactory structural section and the planned cover includes 4 inches of plant mixed surfacing and 5 inches of cement treated base, the total structural thickness may be reduced as follows:

$$(4'' \times 1.32 = 5.28'') + (5'' \times 1.50 = 7.50'') = 12.78'', \text{ or } 13 \text{ inches.}$$

The use of 9 inches of the higher strength treated layers in the upper portion of the structure is equivalent to 13 inches of gravel cover. This results in a 4 inch saving in total thickness in the above example and requires only a 4 inch layer of subbase along with the treated layers to satisfy the design thickness. When considering a roadway structural design estimation, determine the maximum structural section necessary. Then determine whether it would be economically advantageous to use treated base material or to use substitute base or subbase material.

When designing the structural section of roads with a relatively high CBR value, the minimum structural sections recommended by the Corps of Engineers should be observed. These minimum values are shown in Figure 8. There should be a minimum structural section of 6 inches for wheel loads of less than 10,000 pounds, a section of 7.5 inches for loads from 10,000 pounds to 30,000 pounds, and a section of 9 inches for loads greater than 30,000 pounds. The minimum structural section of a road which must support very heavy wheel loads (50,000+ lbs.) has not been adequately determined. A 12 inch minimum structural section should be used to provide a sufficient safety factor.

CHAPTER 7

RIGID PAVEMENT DESIGN

Design Considerations. There is a uniform rational design procedure for the construction of rigid type pavements. These pavements consist of portland-cement concrete slabs resting on a uniformly graded and compacted subbase, or directly upon a compacted subgrade.

In contrast to the flexible type pavement, the rigid type pavement can be designed for any load in conjunction with any prevailing soil conditions, and this design can be rather precisely duplicated at any time at any other location. The flexible pavement design is so dependent upon the soil and construction material types prevalent at each site that any design procedure often becomes a special case.

Rigid pavement supports a load by beam, or slab, action. The concrete slab acts as an elastic structural member. The concrete slab is rigid enough to distribute the imposed load to the supporting subbase in a uniform manner. This stress distribution lowers the maximum stress imposed on the subbase to the point where the bearing support of the soil is not nearly as critical as the bearing support of soils under flexible pavements. The stress imposed on the subbase is seldom greater than 5 - 10 psi.

When designing rigid pavement the subgrade condition and the repetition of design load are taken care of in the design of the concrete slab. Usually, the subbase is composed of an open-graded crushed rock so that the concrete slab will have uniform support and so that there will be adequate subsurface drainage.

Subgrade Modulus. The soil test which is often used to determine the load-supporting value of the native soil and the subbase in place is the plate bearing test. The test is made by placing a circular plate on a level portion of soil which is to be tested. The circular plate, usually of 30-inch diameter, is loaded in order to measure the stress necessary to produce deflection, or displacement, in the soil. This stress-strain relationship is known as the modulus of subgrade reaction, k (9,24). The modulus of subgrade reaction, k, is defined as the reaction of the subgrade per unit of area of deformation and is given in pounds per square inch of area per inch of deformation, or pounds per cubic inch. The unit load for a deformation of 0.05 inch is generally used in determining k. The determination of k is made in the field on the subgrade in place and with an optimum soil moisture content, if possible.

It is possible to make a comparison between the subgrade modulus soil values and the CBR soil values. In this manner it is possible to base rigid pavement design upon CBR soil values which are determined in a soil-testing laboratory. Figure 10 shows the comparison between these two soil-testing methods (47). It must be remembered that this is not a precise relationship but one that may vary as much as 20 per cent or more (52).

Equivalent Single-Wheel Load. In order to design a concrete road it is necessary to determine the maximum equivalent single-wheel

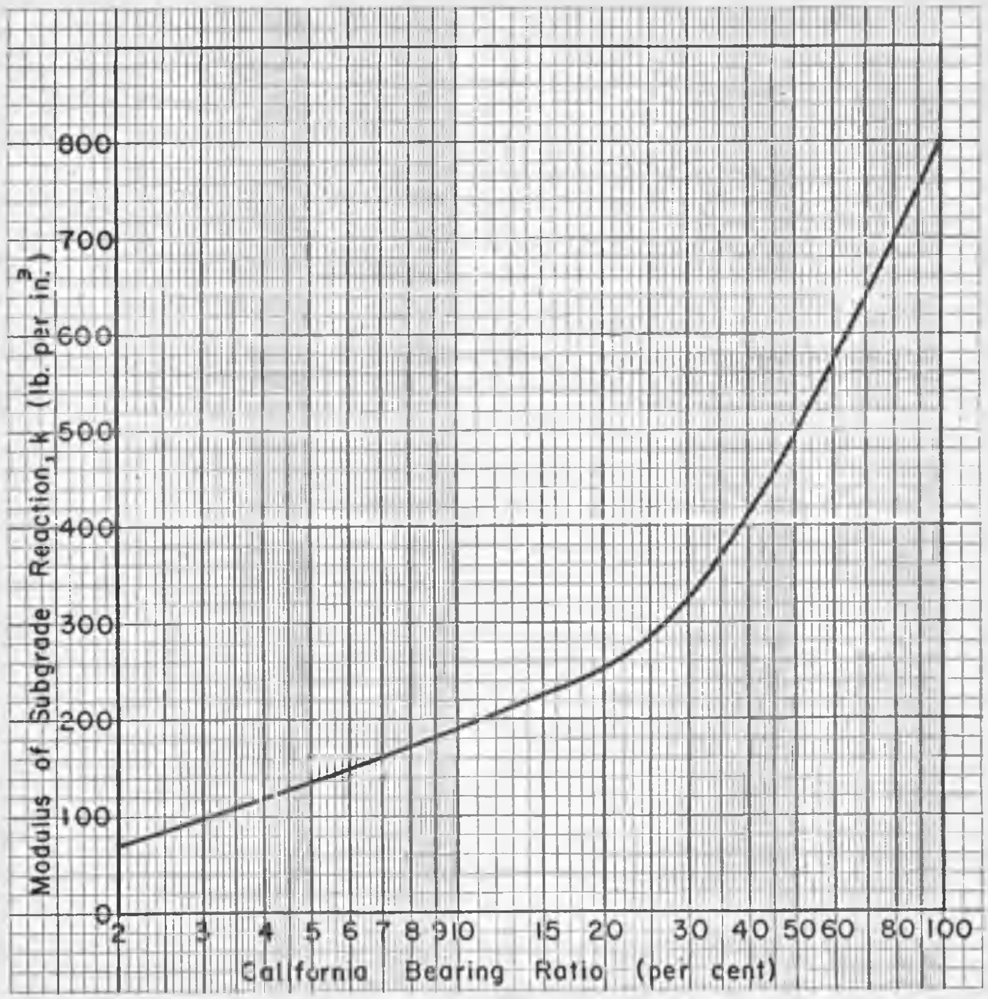

Figure 10. Approximate relationship between the California Bearing Ratio and the modulus of subgrade reaction. (After Portland Cement Association.)

load. This is even more important in designing rigid type pavements than flexible type pavements because the load is distributed over a much larger area and the large areas of stress distribution overlap.

There are different ways to determine this equivalent single-wheel load. Two of the most common methods will be discussed here. The first method to be considered is for dual wheels only (64). The assumption is made that the other wheel spacings are so far apart that the stress distribution interference between wheels is negligible.

Consider a load of 40,000 pounds on dual 18.00-33 tires with a ply rating of 32. Figure 2 shows that the tire pressure should be about 70 psi. The recommended center-to-center spacing of these tires when used as duals is 23.1 inches.

The approximate amount of each tire contact area is 20,000 pounds/70 psi = 285.7 square inches. The radius of contact area of each tire is $(285.7/3.1416)^{\frac{1}{2}} = (90.7)^{\frac{1}{2}}$ or 9.5 inches. It should be noted that Figure 7 shows a larger radius of contact area. This larger radius is given for maximum tire loading.

Thus, the area of contact of the two tires plus space between is 23.1 x 19.0 + (2 x 142.85) = 724.6 square inches. The radius of equivalent contact area is

$$(724.6/3.1416)^{\frac{1}{2}} = (230)^{\frac{1}{2}} = 15.16 \text{ inches.}$$

The equivalent tire pressure for use in design is 40,000 pounds/724.6 square inches equals 55.3 psi. Thus, the equivalent single-wheel load is left at 40,000 pounds, but the equivalent tire pressure of the single wheel which could replace the dual wheel is determined.

The second method of determining the equivalent single-wheel load is used by the Civil Aeronautics Administration. In order to present the CAA method of determining the equivalent single-wheel load (53), it is necessary to present the equation developed by Westergaard (7) for the determination of the radius of relative stiffness.

The equation and the values obtained for different slab thicknesses and subgrade modulus values are listed in Table IX.

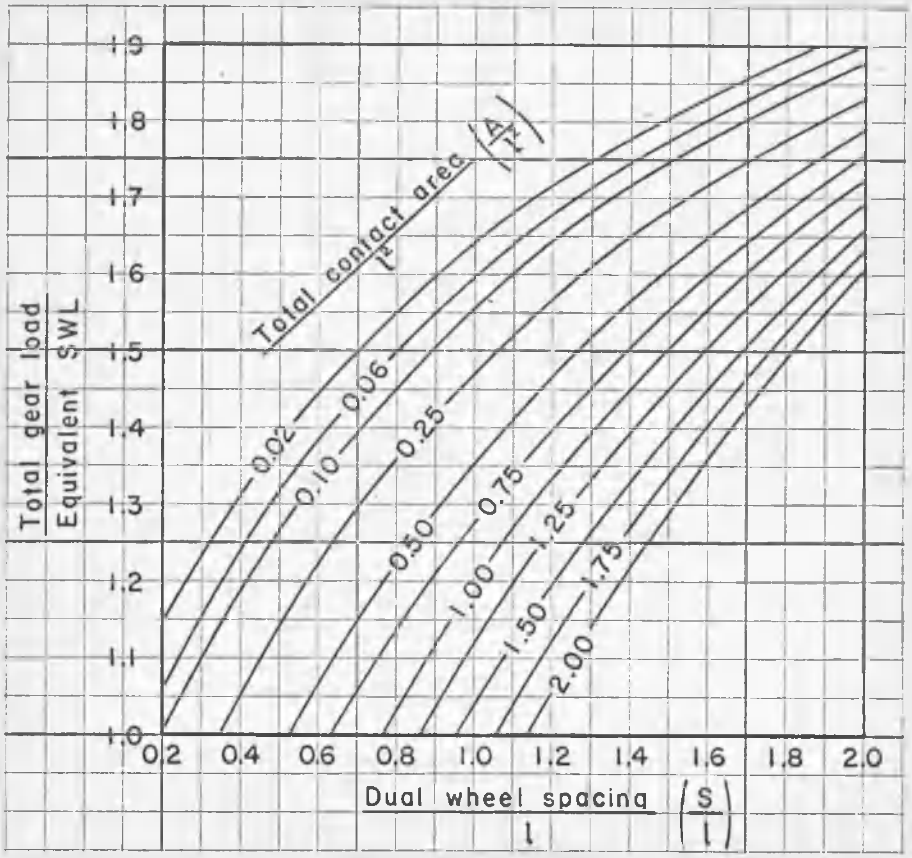

Figure 11. Chart for determination of equivalent single-wheel load for rigid pavements and dual-wheel gear. (After Federal Aviation Agency.)

The dual-wheel conversion chart used by the CAA is shown in Figure 11. To use this chart it is necessary to make a preliminary estimate of the required pavement thickness, h. It is also necessary to compute the total contact area for the two tires which support the imposed load, and the center-to-center spacing of the wheels, S.

In order to illustrate the procedure the equivalent single-wheel

TABLE IX

RADIUS OF RELATIVE STIFFNESS

(μ = 0.15 E = 4,000,000 psi Radius in inches)

d, in.	k = 50	k = 100	k = 200	k = 300	k = 400	k = 500
6.0	34.84	29.30	24.63	22.26	20.72	19.59
6.5	36.99	31.11	26.16	23.64	22.00	20.80
7.0	39.11	32.89	27.65	24.99	23.25	21.99
7.5	41.19	34.63	29.12	26.32	24.49	23.16
8.0	43.23	36.35	30.57	27.62	25.70	24.31
8.5	45.24	38.04	31.99	28.91	26.90	25.44
9.0	47.22	39.71	33.39	30.17	28.08	26.55
9.5	49.17	41.35	34.77	31.42	29.24	27.65
10.0	51.10	42.97	36.14	32.65	30.39	28.74
10.5	53.01	44.57	37.48	33.87	31.52	29.81
11.0	54.89	46.16	38.81	35.07	32.64	30.87
11.5	56.75	47.72	40.13	36.26	33.74	31.91
12.0	58.59	49.27	41.43	37.44	34.84	32.95
12.5	60.41	50.80	42.72	38.60	35.92	33.97
13.0	62.22	52.32	43.99	39.75	36.99	34.99
13.5	64.00	53.82	45.26	40.89	38.06	35.99
14.0	65.77	55.31	46.51	42.02	39.11	36.99
14.5	67.53	56.78	47.75	43.15	40.15	37.97
15.0	69.27	58.25	48.98	44.26	41.19	38.95
15.5	70.99	59.70	50.20	45.36	42.21	39.92
16.0	72.70	61.13	51.41	46.45	43.23	40.88
16.5	74.40	62.56	52.61	47.54	44.24	41.84
17.0	76.08	63.98	53.80	48.61	45.24	42.78
17.5	77.75	65.38	54.98	49.68	46.23	43.72
18.0	79.41	66.78	56.16	50.74	47.22	44.66
19.0	82.70	69.54	58.48	52.84	49.17	46.51
20.0	85.95	72.27	60.77	54.92	51.10	48.33
21.0	89.15	74.97	63.04	56.96	53.01	50.13
22.0	92.31	77.63	65.28	58.98	54.89	51.91
23.0	95.44	80.26	67.49	60.98	56.75	53.67
24.0	98.54	82.86	69.68	62.96	58.59	55.41

Radius of relative stiffness $= l = \sqrt[4]{\dfrac{Ed^3}{12(1-\mu^2)k}} = 24.1652\sqrt[4]{\dfrac{d^3}{k}}$

load of a dual-wheel gear loading of 40,000 pounds will be determined in relation to a rigid pavement 10 inches thick.

h = 14 inches	l = 55.31 inches
S = 23.1 inches	$\frac{S}{l}$ = 0.42
A = 571.4 square inches	$\frac{A}{l^2}$ = 0.19
k = 100 psi per inch	

Enter the horizontal axis, Figure 11, at $S/l = 0.42$ and carry a line vertically to the $A/l^2 = 0.19$. Interpolation is necessary. From this intersection a horizontal line will intercept the vertical axis at 1.13 which is the ratio that the total gear load bears to the equivalent single-wheel load. The equivalent single-wheel load is equal to 40,000 pounds divided by 1.13, or 35,100 pounds.

The first method of computing the equivalent single-wheel load is preferred since it is not necessary to make a preliminary estimate of the required concrete slab thickness.

Conversion of dual-tandem loadings to equivalent single-wheel loads may be accomplished by the use of the curves on Figure 12. The dual-tandem gear introduces an additional factor, S_t/l, in which S_t is the spacing of the axles of the tandem gear. As in the case of dual wheels, S is the center-to-center spacing of the dual tires on an axle. If the spacing of the pairs of fore and aft tires is not equal, the lesser of the two spacings is taken as S.

The equivalent single-wheel load of a dual-tandem gear loading of 161,000 pounds will be determined in relation to a rigid pavement 12 inches thick. For this example:

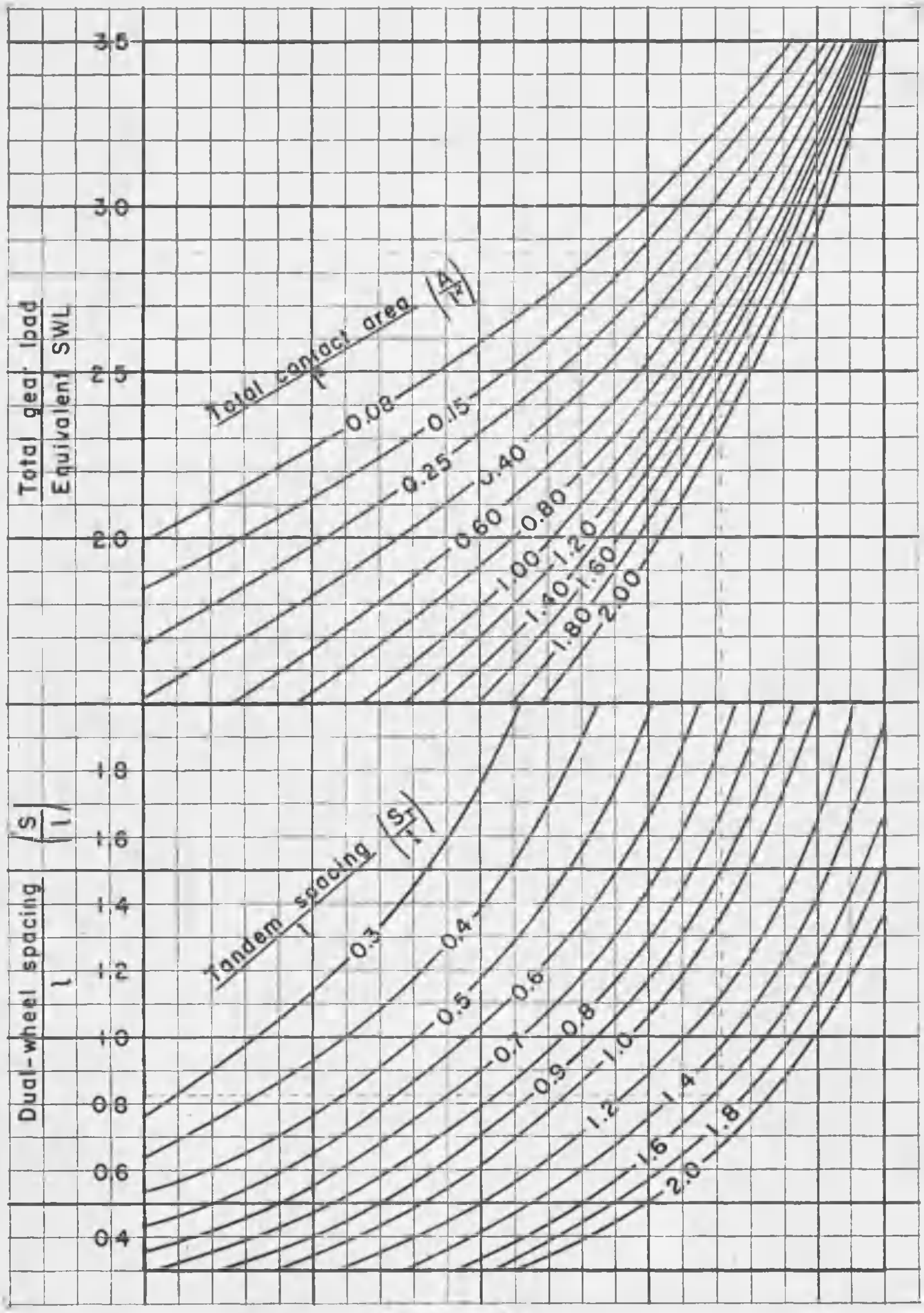

Figure 12. Chart for determination of equivalent single-wheel load for rigid pavements and dual-tandem gear. (After Federal Aviation Agency.)

$h = 12$ inches	$\ell = 37.44$ inches
$S = 31.25$ inches	$\frac{S}{\ell} = 0.83$
$S_t = 61.25$ inches	$\frac{S_t}{\ell} = 1.63$
$A = 824$ square inches	$\frac{A}{\ell^2} = 0.59$

Enter the vertical axis of the lower group of curves, Figure 12, at $S/\ell = 0.83$, and carry a line horizontally to the intersection with the curve representing $S_t/\ell = 1.63$, interpolating when necessary. From this intersection project vertically to intersect the curve representing $A/\ell^2 = 0.59$ in the upper group of curves. A horizontal line from this point will intercept the vertical axis of the upper group of curves at 2.87 which is the ratio that the total gear load bears to the equivalent single-wheel load. In this case the equivalent single-wheel load is equal to the total gear load of 161,000 pounds divided by 2.87, or 56,000 pounds.

The approximate conversion factors to use in determining a preliminary concrete slab thickness value, h, is 1.35 for dual wheels, and 3.00 for dual-tandem wheels.

Safety Factor. The safety factor to be used in designing a rigid type pavement is determined by the number of repetitions of the design load which are expected during the lifetime of the road. Figure 13 shows the relationship between the safety factor and the number of stress repetitions. This safety factor is taken into account by applying it to the modulus of rupture of the concrete. Ordinarily, the concrete mix is proportioned so that the modulus of rupture of the concrete is about 700 psi.

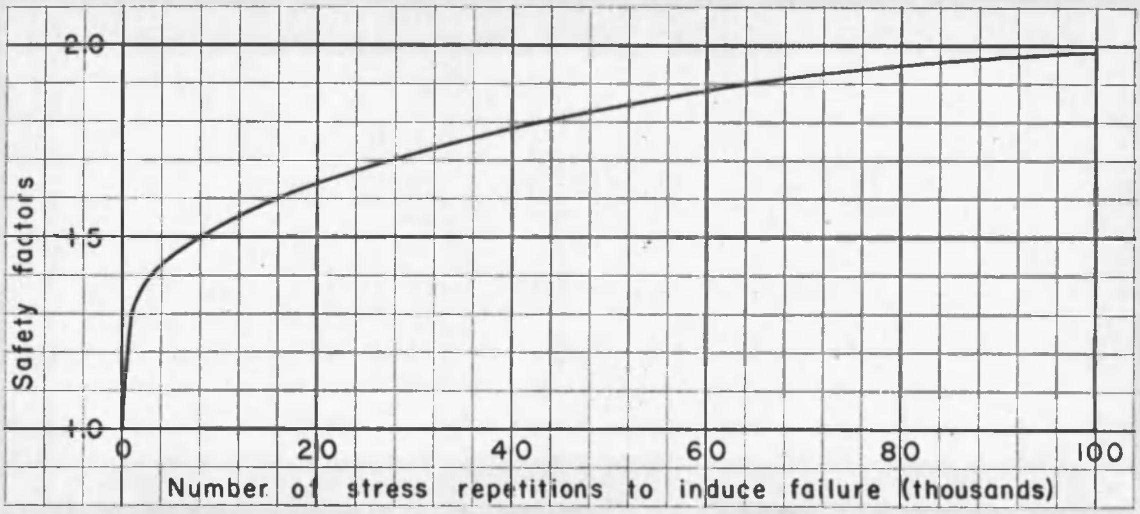

Figure 13. Fatigue of concrete in flexure. (After Portland Cement Association.)

The modulus of rupture of the concrete is the bending strength of the concrete slab. The maximum bending strength is divided by the safety factor which is chosen. If the modulus of rupture of the concrete slab is 700 psi and there are to be over 100,000 repetitions, the safety factor is 2. The maximum stress of 700 psi divided by a safety factor of 2 gives a working bending stress of 350 psi.

Design Curves. The four design curves (Figures 14, 15, 16, 17) are based on semi-empirical formulae developed by Pickett (47).

Case I—Protected Corners: $$S=\frac{3.36P}{d^2}\left[1-\frac{\sqrt{\frac{a}{l}}}{0.925+0.22\frac{a}{l}}\right] \quad \text{(Formula 1)}$$

Case II—Unprotected Corners: $$S=\frac{4.2P}{d^2}\left[1-\frac{\sqrt{\frac{a}{l}}}{0.925+0.22\frac{a}{l}}\right] \quad \text{(Formula 2)}$$

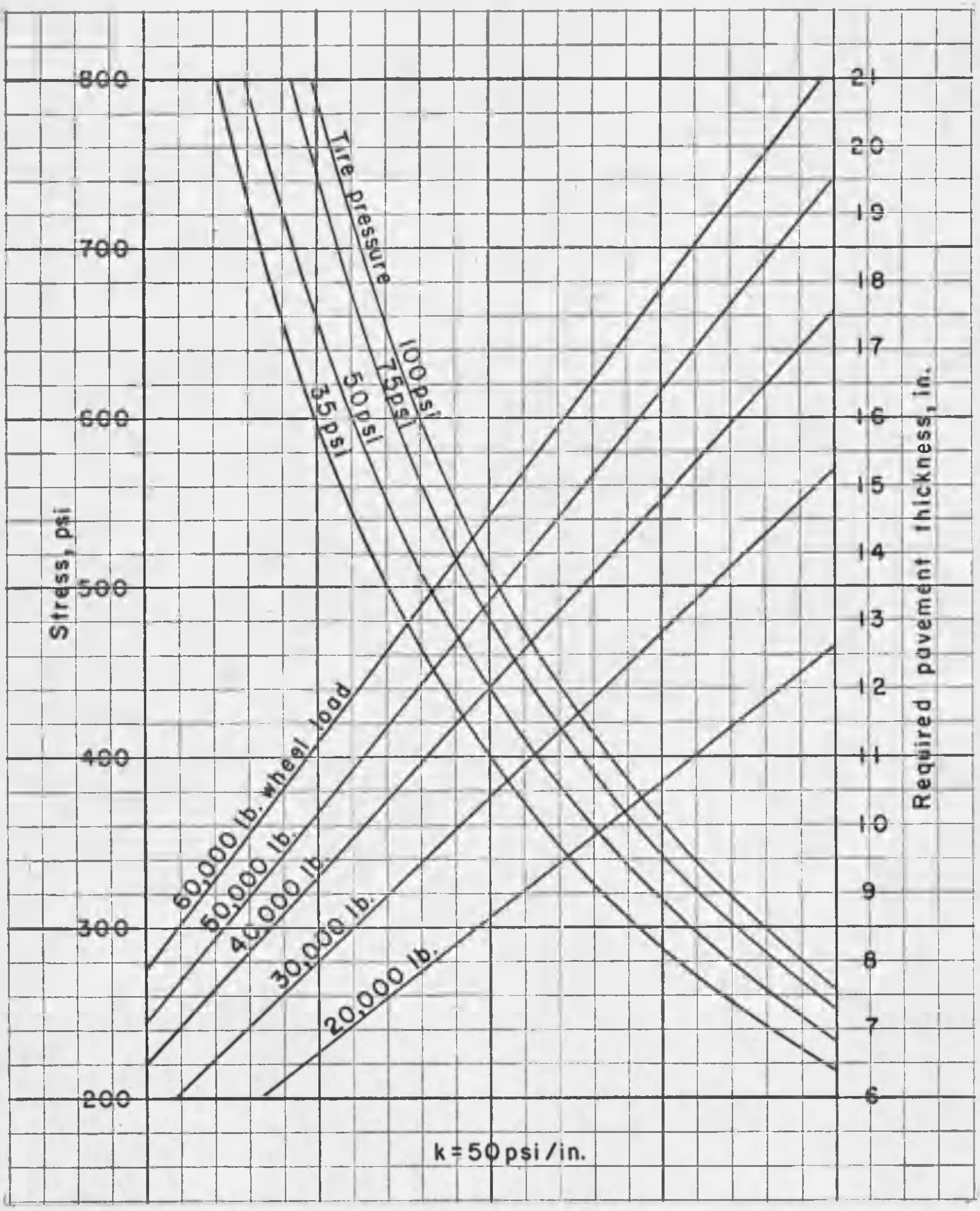

Figure 14. Design chart for portland-cement concrete pavements having protected corners and subgrade modulus of 50 psi/inch. (After Portland Cement Association.)

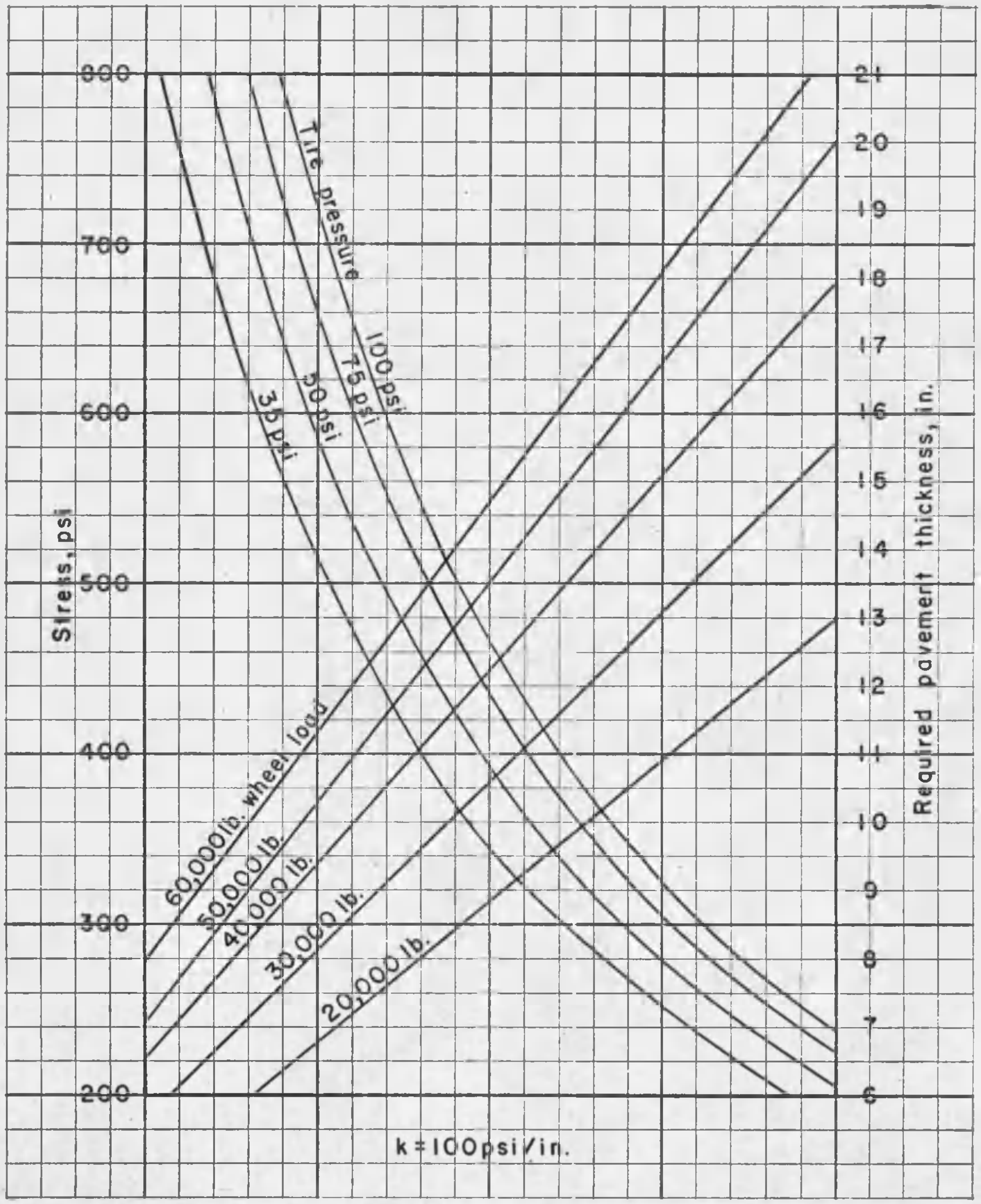

Figure 15. Design chart for portland-cement concrete pavements having protected corners and subgrade modulus of 100 psi/inch. (After Portland Cement Association.)

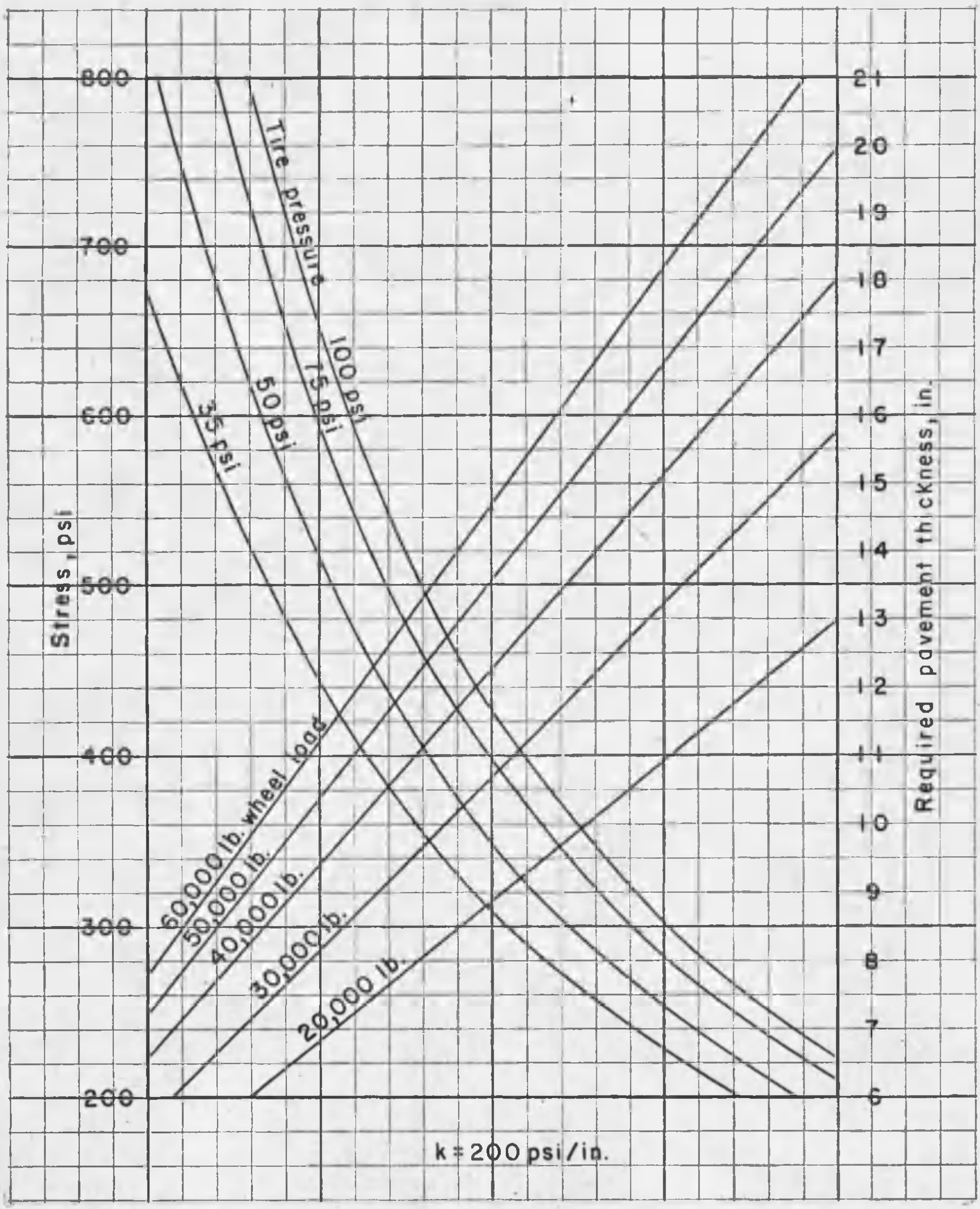

Figure 16. Design chart for portland-cement concrete pavements having protected corners and subgrade modulus of 200 psi/inch. (After Portland Cement Association.)

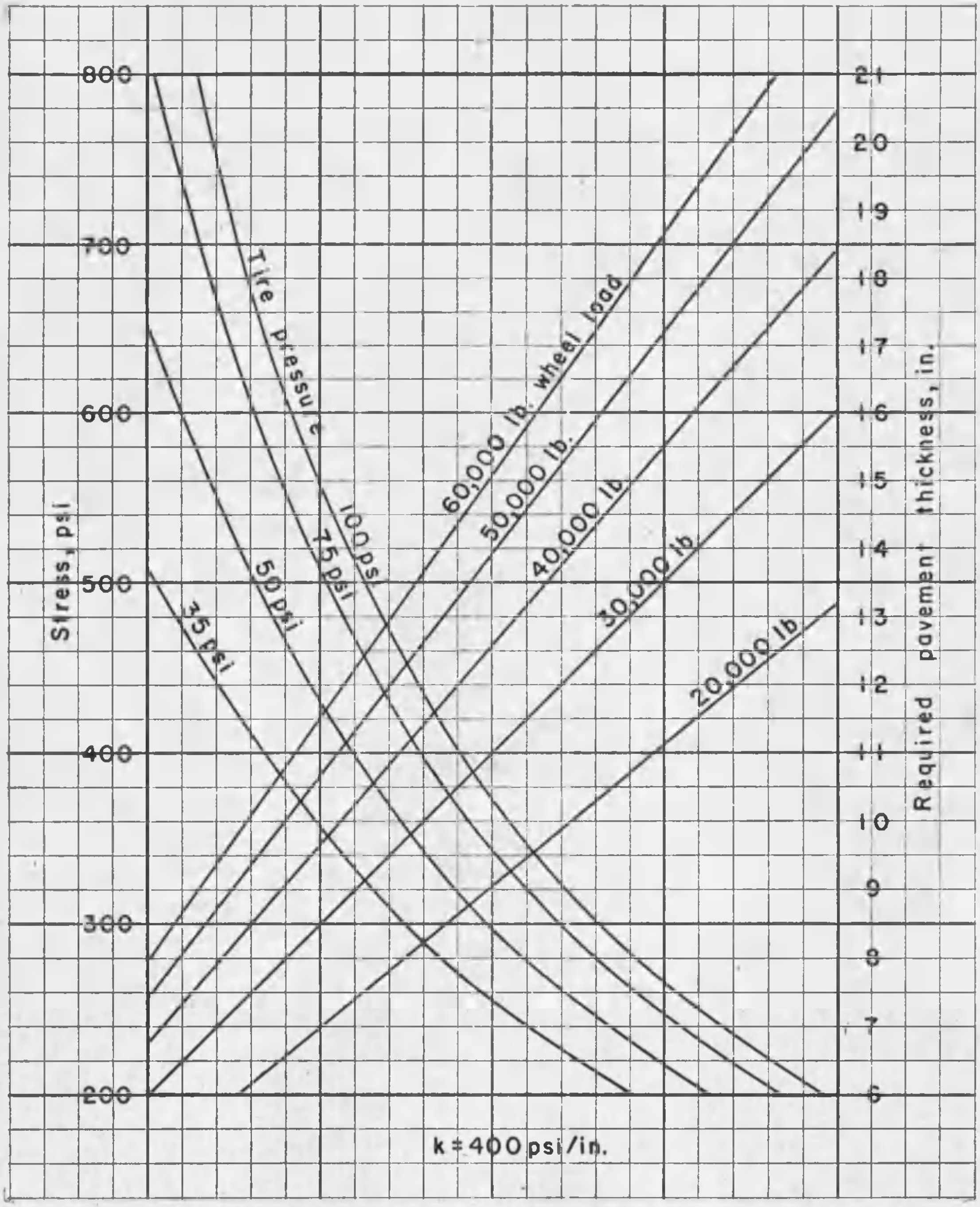

Figure 17. Design chart for portland-cement concrete pavements having protected corners and subgrade modulus of 400 psi/inch. (After Portland Cement Association.)

in which

S = maximum tensile stress in psi at the top of the slab in a direction parallel to the bisector of the corner angle, due to a wheel of P pounds.

P = wheel load in pounds placed on the slab corner in a position of maximum slab strain and increased by an impact factor.

d = thickness in inches of a uniform-thickness concrete slab at a corner.

a = radius in inches of the circular area equivalent to the contact of the tire with the pavement.

l = radius of relative stiffness of the pavement.

The four design curves are for the design of concrete pavements with protected corners. This means that there are load-transfer devices, such as dowels, which transfer at least 20 per cent of a wheel load on a slab corner to the next slab. If, for any reason, no load-transfer devices are used, the stresses determined from the design curves should be increased by 25 per cent (64).

The design curves are made for four different conditions of subgrade modulus. If interpolation is necessary compute the design thickness for the chart, or charts, nearest the measured subgrade modulus, k, and interpolate between the values derived.

The following example is given to show how the design charts are used. The design chart (Figure 14) is read by entering it at the left side at the working stress (350 psi) and moving horizontally until the proper tire pressure (50 psi) is intersected; then move vertically upward until the gross controlling wheel load (40,000 lbs.) is intersected; thence, horizontally to the right where the value of the indicated d in in inches (14.0 inches) is read.

Subbase Requirements. The amount and quality of subbase required is largely determined by climatic and topographic conditions. When there is a poor subgrade the required thickness may be greater. Table X is a list of average required subbase thicknesses for different CBR ratings and loads (53). If the drainage is good and the frost conditions are not excessive, the thickness of subbase required is considerably less than the values listed. The subbase should have a minimum thickness of 4 inches.

TABLE X

REQUIRED SUBBASE THICKNESS IN INCHES

Single-wheel load (kips)	CBR- 2 - 3	3 - 5	5 - 25	25 - 80
10 - 40	12	8	6	4
40 - 60	12	12	8	4
60 - 100	16	12	8	6

The subbase should consist of open-graded material with a CBR of 80 - 100 (crushed rock). The required subbase compaction should be 95% of maximum density and the 4 - 6 inches of the top of the subgrade should be 80% of maximum density. When the subgrade consists of plastic materials or clays which have a large swell factor, the upper 2 - 4 inches of the subbase, or subgrade, may consist of cement treated material.

Subbase thickness may not be substituted for slab thickness.

Figure 18 shows the increase in subgrade modulus, k, which is acquired by adding a subbase with a CBR of 80+. A subbase thickness of 12 inches is not equivalent to even one inch of concrete slab thickness (49,64).

The subbase used should be open-graded material so that there is good drainage and so that, consequently, there will not be 'pumping' of water and fines (65). This 'pumping' of the concrete slab by the repetition of heavy loads will eventually destroy the subbase layer by displacement and cause the concrete slab to break up.

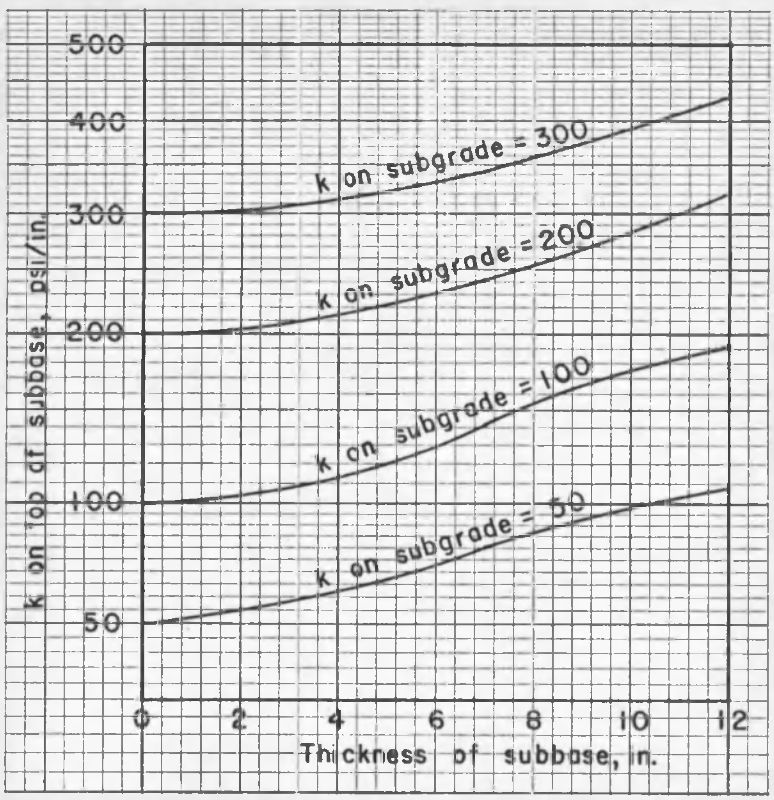

Figure 18. Effect of various subbase thicknesses on 'k' value. (After Portland Cement Association.)

The cost of using steel reinforcement in concrete slabs was discussed in Chapter 4. The use of reinforcing steel in concrete road construction is a very expensive procedure. Further research is needed in the use of steel reinforcing in very heavy design roads. The steel does not increase the concrete slab strength in the amount that it is ordinarily used. The reinforcing tends to hold the concrete sections together as the concrete slab cracks into sections from 5 to 15 feet long (34, 35, 36).

Sample Design Computations. Consider the same tires and loading which was used in the equivalent single-wheel computations. The load is 40,000 pounds on dual 18.00-33 tires with a ply rating of 32. The tread is 78 inches center-to-center of dual tires, the subgrade is a medium plastic clay with a CBR of 4, and over 100,000 repetitions of stress. The expected modulus of rupture of concrete is 650 psi at 28 days, so the allowable stress is 325 psi.

The radius of equivalent contact area was found to be 15.16 inches and the equivalent tire pressure for use in design was found to be 55.3 psi.

Reference to Figure 10 indicates that for a CBR of 4, the modulus of subgrade reaction, k, would be 125 psi per inch. A 4-inch granular subbase would be recommended on this type of soil, and consequently the k value used in the design should be increased, as Figure 18 indicates, to 140 psi per inch.

From Figure 15 (k = 100 psi/in.), for a stress of 325 psi the required pavement thickness is approximately 14 inches, and from Figure 16 (k = 200 psi/in.), it would be about 13 inches. A pavement of 14 inches thickness would be suitable for the design load if the k value of the subgrade is 140 psi/in.

Check these values by computing the stress for case 1 - protected corners, where, interpolating from Table IX, l = 50.85 inches, a = 15.16 inches, d = 14 inches, and P = 40,000 pounds. 'S' is computed to be 308 psi. Therefore, the 14 inch slab is adequate.

Dowels. There are different methods of constructing slab transverse joints in concrete roads. There is still much experimentation in this phase of rigid pavement design (66,67). There are three different methods of constructing concrete roads. The concrete may be continuously poured so that there are no open transverse joints (34,35,36); The transverse joints may be constructed with no load-transfer devices; the transverse joints may be constructed with load-transfer devices, such as dowels (68,69), tongue-in-groove joints, or 'sleepers'. 'Sleepers' are subgrade concrete slabs beneath the joints upon which the surface slabs rest.

There are two types of joints, contraction joints and expansion joints. The continuously-poured concrete roads do not have expansion joints since they are found to be unnecessary. Contraction joints are necessitated by the setting of the concrete and temperatures lower than the temperature of the concrete when it was poured in place. Expansion joints are constructed to allow for expansion of the concrete in temperatures greater than the temperature of the concrete when it was poured in place. Continuously-poured concrete is found to be able to absorb the compressive stress caused by expansion due to higher temperatures. It is considered that an imperceptible bowing of the pavement over a large distance relieves this pressure.

Expansion joints are usually constructed by forms when pouring the concrete. Sometimes contraction joints are constructed in the same way. It is now common practice to saw a transverse groove in the concrete slab, at intervals of 10 - 25 feet, to a depth of 1 - 2 inches. This causes a weakness plane in the slab. When a superimposed load is introduced the concrete slab cracks apart at this weakness plane. The crack is somewhat irregular and so tends to support the load while also acting as an expansion joint. When dowels are set in the concrete beneath the sawed groove the joint is referred to as a dummy groove contraction joint (47). It is expected that the concrete slab will be cracked into sections from 5 - 10 feet in length after several years' use. Heavier design concrete pavements may not crack at such close intervals. By overdesign this cracking can be controlled, but it is not economically feasible.

Since the load-transfer device known as a 'sleeper' does not work well over a period of time, due to irregular settlement, it is not recommended for use.

Dowels are presently the most economical method of transferring a load from one concrete slab to the next. It is advisable to set the dowels in close-fitting tubes, which are set in the concrete when it is poured. The joints will open and close as the temperature changes, so the dowels should be close-fitted but free moving. It may be necessary to purchase non-corrosive dowels, especially around a mine where leaching is practiced (36).

The dowel system necessary to transfer at least 45 per cent of the edge load across the joint is determined by trial. From Figure 19 the capacity factor for dowels spaced at 12 inch centers in a slab having an l of 50.85 inches is 4.3.

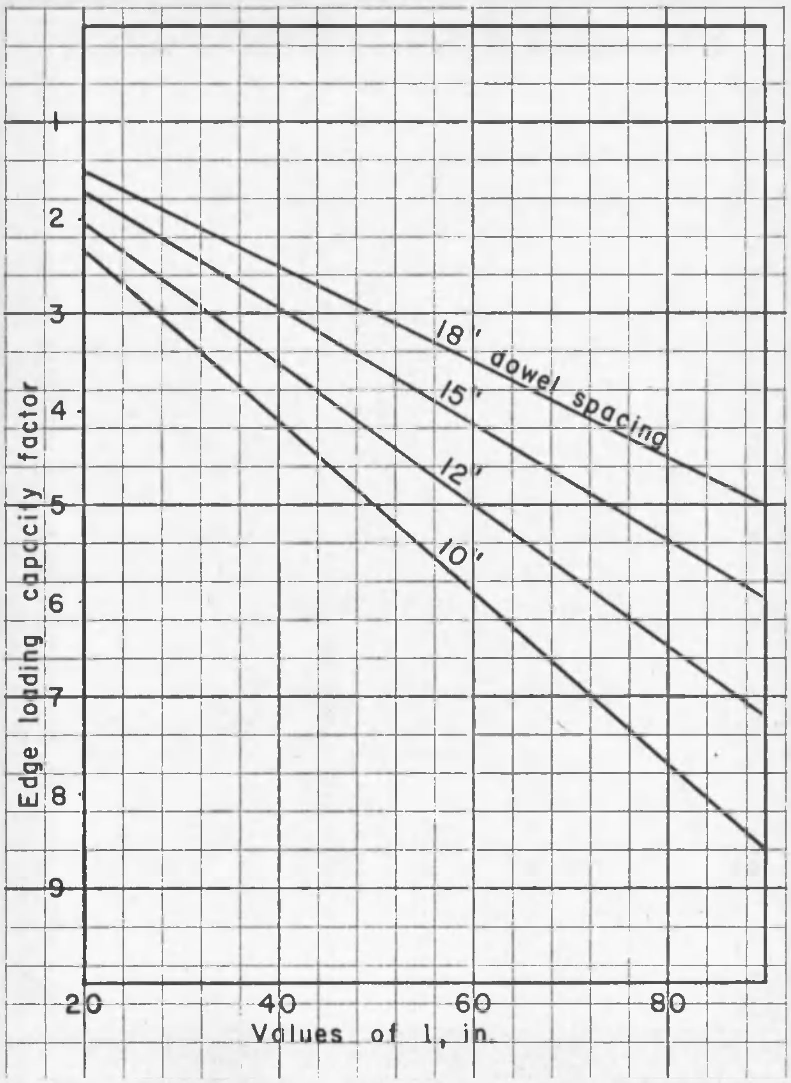

Figure 19. Determination of total capacity factor of a dowel system. (After Portland Cement Association.)

The ratio of the center-to-center spacing of wheels to l = 78 inches/50.85 inches = 1.53. Since this is less than 1.8 there will be overlapping of dowel stress from the two wheels and, as shown in Figure 20, the allowable load-transfer capacity of the dowel system should be reduced to 0.86 x 4.3 = 3.70 .

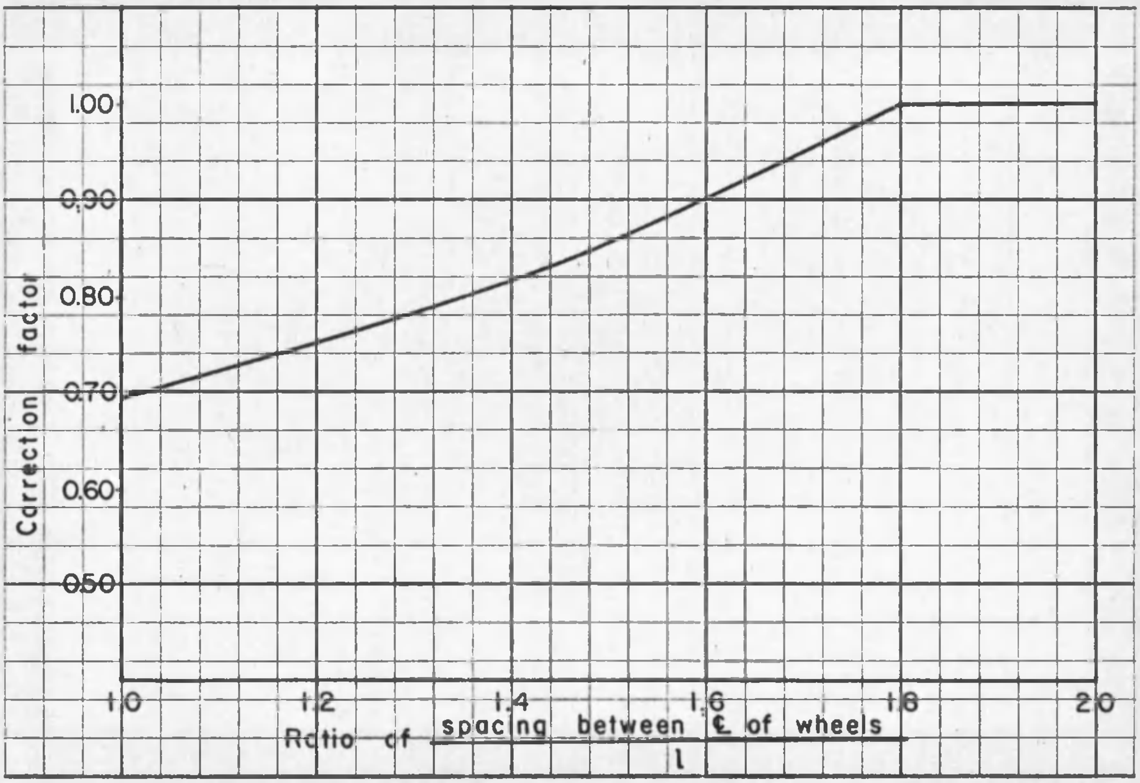

Figure 20. Determination of correction factors for the load-transfer capacity of dowels. (After Portland Cement Association.)

Figure 20 is based upon the equation relating the capacity factor to the distance from the load to the dowel.

$$\text{Capacity factor} = \frac{(1.8\, l - \text{distance of dowel from center of load})}{1.8\, l}$$

For this allowable capacity factor the required load-transfer capacity of each dowel, if 45 per cent of the load is to be transferred, is found to be 0.45 x 40,000/3.70 = 4,865 pounds.

From Figure 21 it is seen that a 1 5/8-inch dowel will transfer

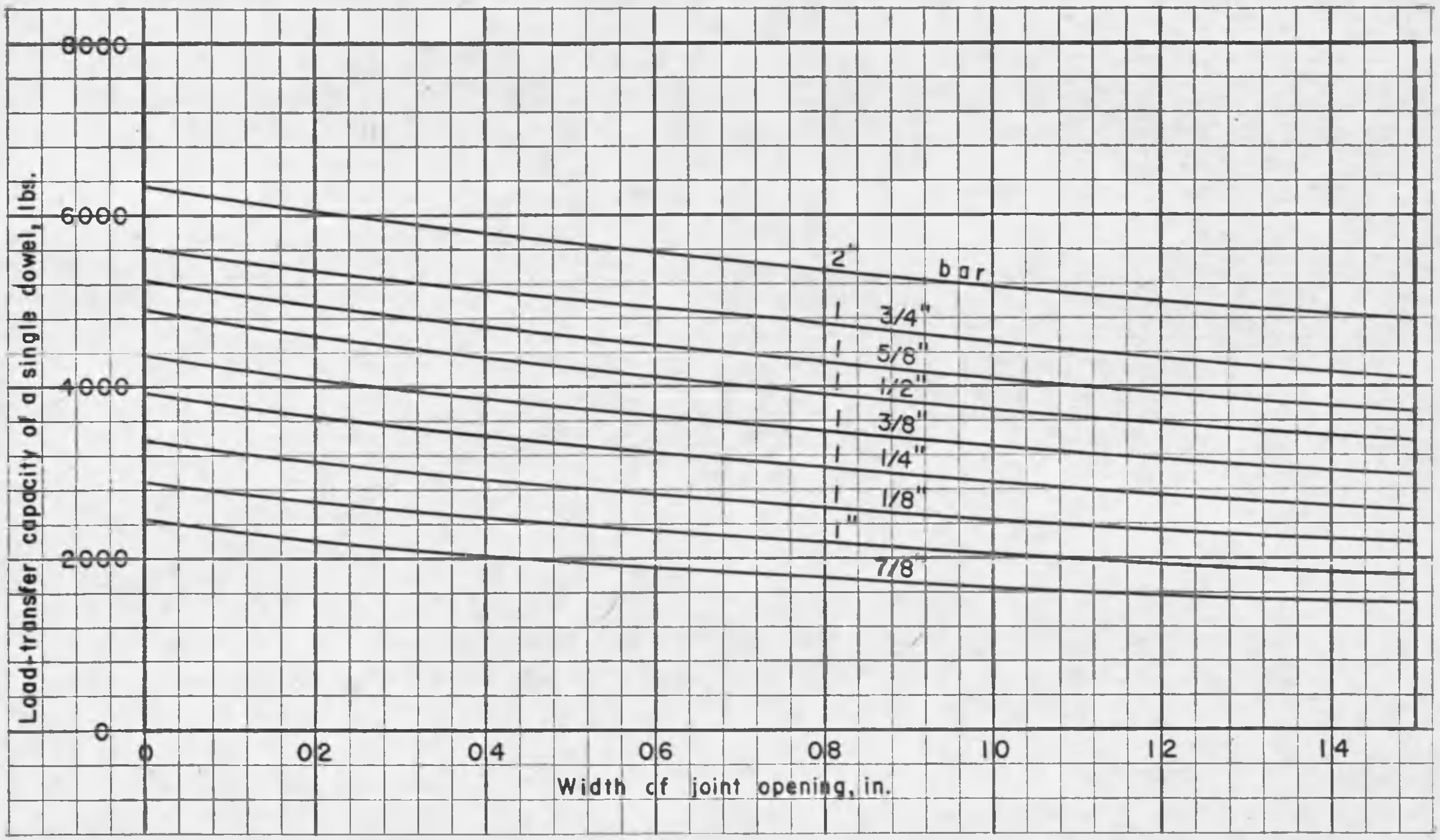

Figure 21. Load-transfer capacities of single dowels for joint openings of various widths. (After Portland Cement Association.)

4,900 pounds across a 0.25-inch joint. Dowels of this size will therefore be adequate for contraction joints if they are spaced at 12-inch centers. For expansion joints, assuming a 0.75-inch opening, 1 3/4-inch dowels should be used.

These sizes conform to the requirements for protected corners and ensure that the dowels will not be overstressed by the design load.

The use of dowels is recommended in the construction of concrete slab joints at 20 to 40 foot intervals. If contraction joints are needed they should be cut in the manner prescribed.

There have been experimental concrete roads constructed with the joints constructed, or cut, on an angle instead of normal to the sides of the slab (36). This is done so that only one wheel load at a time would be crossing the joint. This causes a longer joint to be constructed but increases the wearing life of the pavement since most of the destruction of a concrete pavement is centered around the joints.

It is not recommended that a continuously-poured concrete pavement be constructed unless enough deformed-bar steel reinforcing is used to hold the slabs together as the traffic breaks the slabs into sections. As previously mentioned, the reinforcing-steel bars hold the edges of the cracks tightly together but do not increase the load-carrying capacity of the concrete slab (70). This minimum amount of steel, as stated in Chapter 4, is longitudinal steel in the amount of 0.5% of the transverse cross-sectional area of the slab, and transverse steel sufficient to resist the tensile stress in the concrete caused by contraction upon setting and curing.

The pavement should be poured in two equal lanes and joined by tongue-in-groove molding. Tie bars across the transverse joints should be unnecessary. Where tie bars are required they should consist of deformed steel bars (53,70). The bars should be 5/8-inch in diameter and 30 inches long, and should be spaced 30 inches on center.

CHAPTER 8

CONCLUSIONS

<u>Summary</u>. None of the factors of roadbed design and construction have been completely investigated. Each facet of the design procedure is still being investigated in an attempt to gain new knowledge and to improve the design of the roads which are to be built.

The Highway Research Review (<u>67</u>) gives an insight into the voluminous research projects which are being undertaken. Each year more aspects of roadway construction are investigated as the roadway construction demands become larger and more varied.

The construction of well-paved pit haulage roads is shown to be economically advantageous where the roads are expected to provide service for a large amount of haulage for several years. The high costs of truck tires and haulage over poor roads with high rolling resistance and a high traction loss outweigh the initial construction costs of more adequate pit roads.

The design procedures which have been discussed in this manuscript are considered adequate for the construction of roads which will provide suitable service for modern heavy-duty trucks.

The design of rigid pavements has been most thoroughly investigated so that there can be little further change expected in this field of highway research. The further developments will involve smaller details such as joints, reinforcement, materials comparison, and soil treatment.

There will be many changes made in the design procedures for the construction of flexible pavements in the next few years. As more heavy-design roads are constructed, it will be possible to enact more definite rules to follow in further roadway design. There is not now a truly rational design procedure for the construction of flexible pavements.

Further Research. There are many phases of pit road design which would bear investigation. Some of the questions of immediate importance are: the design of low-type and intermediate-type roads; the effect of haulage losses due to varying grades on different types of roads; an adequate comparison between the costs of construction and use of different road types versus railroad haulage.

It is hoped that the material provided in this manuscript will be of use in the further investigation of pit haulage systems.

BIBLIOGRAPHY

1. Harris, F. A. "Selection and Design of Semi-Flexible and Conventional Type Pavements," Proceedings, Highway Research Board, Vol. 35, 1956.

2. Woods, K. B. (ed.) "Thickness Design Methods for Flexible Pavements," Highway Engineering Handbook, Part 4. McGraw-Hill Co., 1960.

3. Burmister, D. M. "The Theory of Stresses and Displacements in Layered Systems and Application to the Design of Airport Runways," Proceedings, Highway Research Board, Vol. 23, 1943.

4. "Progress Report of Subcommittee on Methods of Measuring Strength of Subgrade Soil - Review of Methods of Design of Flexible Pavements," Proceedings, Highway Research Board, Vol. 25, 1946.

5. "The WASHO Road Test, Part 2: Test Data, Analysis, and Findings," Highway Research Board Special Report 22, 1955.

6. "The AASHO Road Test: History and Description of the Project," Highway Research Board Special Report 61A, 1961.

7. Westergaard, H. M. "Computation of Stresses in Concrete Roads," Proceedings, Highway Research Board, Vol. 5, Part 1, 1925.

8. "Design Practices, The Truck Weight Problem in Highway Transportation," Highway Research Board Bulletin 26, Chapt. 5, 1950.

9. Soils Manual for Design of Asphalt Pavement Structures, The Asphalt Institute, Oct., 1961.

10. Sherman, G. B. "Recent Changes in the California Design Method for Structural Sections of Flexible Pavement," Materials and Research Department, Division of Highways, Department of Public Works, State of California, Mar., 1958.

11. Porter, O. J. "Foundations for Flexible Pavements," Proceedings, Highway Research Board, Vol. 23, 1943.

12. Foster, C. R., and Ahlvin, R. G. "Notes on the Corps of Engineers' CBR Design Procedure," Highway Research Board Bulletin 210, 1959.

13. "Axle-Load and Gross-load Trends," Public Roads, Journal of Highway Research, Vol. 25, No. 12, Feb., 1950.

14. "Structural Design of the Roadbed," Section 7-600 revised Aug. 26, 1957, Planning Manual, Division of Highways, Department of Public Works, State of California, 1962.

15. "Truck Specifications, and Heavy-Duty and Off-Highway Trucks," Automotive Industries, Chilton Co., Vol. 126, No. 6, Mar. 15, 1962.

16. "State Legal Maximum Limits of Motor Vehicle Sizes and Weights Compared with AASHO Standards," Public Roads, Journal of Highway Research, Vol. 30, No. 11, Dec., 1959.

17. "New Products Digest, and Modern Haulage Equipment," Engineering and Mining Journal, McGraw-Hill Publishing Co., Aug., 1962.

18. Stephenson, H. K. "Determination of Permissible Vehicle Weights on Bridges of H Loading Design," Proceedings, American Association of State Highway Officials, 35th Annual Meeting, 1949.

19. Grumm, F. J. "Designing Foundation Courses for Highway Pavements and Surfaces," California Highways and Public Works, Mar., 1942.

20. Drake, W. B., and Havens, J. H. "Re-Evaluation of Kentucky Flexible Pavement Design Criterion," Discussion by S. M. Fergus, Highway Research Board Bulletin 233, 1959.

21. Farrell, F. B., and Paterick, H. R. "Life Characteristics of Highway Surfaces," Public Roads, Journal of Highway Research, Vol. 25, No. 9, 1949.

22. Gronberg, G. D., and Blosser, N. B. "Lives of Highway Surfaces - Half Century Trends," Proceedings, Highway Research Board, Vol. 35, 1956.

23. "Economics of Asphalt and Concrete for Highway Construction," Stanford Research Institute, Menlo Park, California, Mar., 1961.

24. "PCA Soil Primer," Portland Cement Association, Publication SC10-2, Chicago, Ill., 1956.

25. "Standard Specifications," County of Los Angeles Road Department, Los Angeles, California, Nov., 1961.

26. Hveem, F. N., and Carmany, R. M. "The Factors Underlying the Rational Design of Pavements," Proceedings, Highway Research Board, Vol. 28, 1948.

27. Baker, R. F. "Fundamental Factors in Incremental Cost Studies of Pavements," Proceedings, Highway Research Board, Vol. 36, 1957.

28. Stern, E. L. "A New Base for the Highway Construction Bid Price Index Compiled by the Bureau of Public Roads," Public Roads, Journal of Highway Research, Vol. 31, No. 10, Oct., 1961.

29. Engineering News-Record, Vol. 169, No. 14, McGraw-Hill Publication, New York, Oct. 4, 1962.

30. Seelye, E. E. (ed.) Data Book for Civil Engineers, Volume Two - Specifications and Costs, Third Edition, John Wiley and Sons, Inc., New York, 1957.

31. "Comparative Pavement Costs on Interstate-Defense and Primary Roads," Information Series No. 100, The Asphalt Institute, May, 1961.

32. "Clear Facts about Pavements in Arizona," The Asphalt Institute, July, 1959.

33. Stanton, T. E. "Reports on Experiments with Continuous Reinforcement in Concrete Pavements, California," Proceedings, Highway Research Board, Vol. 30, 1951.

34. "Continuous Steel Reinforcement for Experimental Concrete Pavements," Highway Research Board Bulletin 214, 1959.

35. "Experimental Continuously-Reinforced Concrete Pavements: Progress Reports -- 1959," Highway Research Board Bulletin 238, 1960.

36. "Concrete Pavement Design and Performance Studies: 1960," Highway Research Board Bulletin 274, 1960.

37. Bishop, H. K. (chairman) "Report of Committee on Maintenance Costs," Proceedings, Highway Research Board, Vol. 22, 1942.

38. Radzikowski, H. A. (chairman) "Progress Report of the Project Committee on Maintenance Costs," Maintenance Costs, Highway Research Board Bulletin 29, 1950.

39. Radzikowski, H. A. (chairman) "Report of the Committee on Maintenance Costs," Highway Maintenance Studies, Highway Research Board Bulletin 155, 1957.

40. Harmon, J. P. "Use of Lignin Sulfonate for Dust Control on Haulage Roads in Arid Regions," Bureau of Mines Information Circular 7806, United States Department of Interior, Oct., 1957.

41. Drevdahl, E. R. Profitable Use of Excavation Equipment, Technical Publications Desert Laboratories, Inc., Tucson, Ariz., 1961.

42. "Fundamentals of Earthmoving," Sales Training Division, Caterpillar Tractor Co., Peoria, Ill., 1956.

43. 1962 Yearbook, The Tire and Rim Association, Inc., Akron, Ohio, 1962.

44. "The AASHO Road Test: Special Studies," Highway Research Board Special Report 61F, 1962.

45. Freeman, R. A., and Porter, O. J. "Flexible Pavement Test Section for 300,000 Pound Airplanes, Stockton, California," Proceedings, Highway Research Board, Vol. 25, 1946.

46. Paxson, G. S. "Factors Influencing the Stress in Concrete Pavement from Applied Loads," Proceedings, Highway Research Board, Vol. 25, 1946.

47. "Concrete Pavement Design," Portland Cement Association, Publication R18-2, Chicago, Ill., 1951.

48. "Roads and Airfields," Technical Manual 5-250, United States Department of Army, Washington, D. C., 1957.

49. "Design of Concrete Airport Pavement," Portland Cement Association, Publication R147-2, Chicago, Ill., 1955.

50. Palmer, L. A. "Flexible Pavement Design for Airfields," an unpublished manuscript, The Asphalt Institute, circa 1961.

51. "Developing a Set of CBR Design Curves," Appendix A of Instruction Report 4, United States Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss., Nov., 1959.

52. "Design Manual - Civil Engineering," NavDocks DM-5, Department of the Navy, Bureau of Yards and Docks, Washington, D. C., Feb., 1962.

53. "Airport Paving," United States Department of Commerce, Civil Aeronautics Administration, Washington, D. C., Oct., 1956.

54. Yoder, E. J. Principles of Pavement Design, John Wiley and Sons, Inc., 1959.

55. Howe, D. R. "The Hveem Stabilometer and Its Application to Soils in the Structural Design of Pavement Sections," Materials and Research Department, Division of Highways, Department of Public Works, State of California, Mar., 1961.

56. "Methods of Calculating the Design Thickness of Pavement Sections Based on Stabilometer and Expansion Pressure Measurements," Part VII of Test Method 301-B, revised Sept. 1, 1957, Materials Manual, Division of Highways, Department of Public Works, State of California, 1962.

57. "Example Showing the Application of the Revised California Stabilometer Method Design for Soil-Cement," an unpublished manuscript, Portland Cement Association, Soil-Cement Bureau, April, 1958.

58. "Compaction of Embankments, Subgrades, and Bases," Highway Research Board Bulletin 58, 1952.

59. "Thickness Design - Asphalt Pavement Structures for Highways and Streets," The Asphalt Institute, Sixth Edition, April, 1962.

60. "Soil Density and Stability," Highway Research Board Bulletin 93, 1954.

61. Krupp, L. M. "Asphalt Mixed Surface Mat for Heavy-Duty Ore Haulage Road at Silver Bell Mining Properties of American Smelting and Refining Company," paper given at meeting of Arizona section of Open Pit Division, A. I. M. E., April 23, 1956.

62. Herner, R. C. "The Load Transmission Test for Flexible Pavement and Base Courses," Technical Digest (Bureau of Yards and Docks) NavDocks P-23, No. 67, June, 1956.

63. "Soil-Cement Construction Handbook," Portland Cement Association, Publication SC3-7, Chicago, Ill., 1956.

64. "Design of Concrete Pavement for Heavy Wheel Loads," Concrete Information, Highways and Municipal Bureau, Portland Cement Association, Publication HB 24, Chicago, Ill., 1957.

65. "Performance of Granular Subbases under Concrete," Highway Research Board Bulletin 202, 1958.

66. Van Breemen, W. "Experimental Dowel Installations in New Jersey," Proceedings, Highway Research Board, 1955.

67. Joints, Highway Research Review, Number 5, An Annotated Census of Highway Research Projects Current or Completed Since January 1958, Jan., 1962.

68. "Concrete Pavement Design Research 1958," Highway Research Board Bulletin 217, 1959.

69. "Concrete Pavement: Subbase and Joint Construction," Highway Research Board Bulletin 229, 1959.

70. "Concrete Pavement Manual," Suggested Practices for Office and Field, Portland Cement Association, Fifth Edition, Publication R2-5, Chicago, Ill., 1955.