HIGHWAY RESEARCH BOARD

Research Report 16-B

Design of Flexible Pavements

PRESENTED AT THE
Thirty-Third Annual Meeting
January 12-15, 1954

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HIGHWAY RESEARCH BOARD

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DEPARTMENT OF HIGHWAY DESIGN

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Principal Highway Engineer, Bureau of Public Roads

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Foreword

In the past 10 or 15 years, the Committee on Flexible Pavement Design under the guidance of its chairman, A. C. Benkelman, has been one of the most-active working committees of the Highway Research Board. A progress report of its activities would list several high points among its contributions to advancing the design of flexible pavements.

As a result of its work, Wartime Road Problems No. 8 was published in 1943 under the title, "Thickness of Flexible Pavements for Highway Loads". This publication summarized current practice in the United States as of that time, presented basic definitions and gave the range of pavement thickness as related to soil classification.

After the war, this committee continued its work acting as the clearing house for the various state highway departments in formulating methods for use in the various states. This program culminated in 1949 in the revision of the Wartime Road Problems booklet as Current Road Problems No. 8-R, "Thickness of Flexible Pavements." This revised and enlarged publication brought the older one up to date as to definitions, pavement-thickness practice, and soil classification. Sections were added on methods adopted by several state highway departments, a brief description of various thickness formulas that had gained some recognition and a presentation of methods adopted by several governmental agencies including the U. S. Engineer Department, U. S. Navy Department, and Canada.

The committee has held regular meetings each year at the annual meeting of the Highway Research Board and these have been the occasion for concentrated review and discussion of flexible pavement design problems. Each year, there have been papers presented on developments in flexible pavements under the auspices of the committee, and many of these papers have been important contributions in this field. In addition, the committee has held summer meetings at Ann Arbor (Michigan) in 1948, Kansas City (Missouri) in 1950, Lexington (Kentucky) in 1951 and Malad City (Idaho) in 1953. These summer meetings have provided the opportunity for members to see the work being done in these several states through field inspection trips as well as to expedite important committee business. The Board has recently distributed Bulletin 80, "Flexible-Pavement Design," which presents the results of a nation-wide survey by this committee of the methods currently being used by the state highway departments.

The symposium which has been arranged for this year's annual meeting should mark another milestone in the committee's progress. At this time, six of the states are ready to report the results of a review of the experience of the past few years with the methods of design which they adopted through the result of their association with the work of this committee. These papers will, in general, report the results of pavement-performance surveys of roads built in accordance with the several methods of design selected and attempt to evaluate the results as a measure of the adequacy of methods themselves. Comparisons are also given with other roads built without the same design control.

W. S. Housel
Professor of Civil Engineering,
University of Michigan

Contents

Triaxial Tests in Analysis of Flexible Pavements

CHESTER McDOWELL, Senior Soils Engineer,
Texas Highway Department

A previously published method of comparing strengths of soils, as measured by triaxial tests, to estimated wheel-load stresses is discussed with respect to subsequent revisions in testing procedure and methods of interpretation. In order to show how this method of pavement analysis correlates with service behavior of existing pavements, ten projects in south, central, and west Texas were investigated. The investigation included: service behavior, age, traffic loads, thickness of pavement layers, triaxial tests, soil constants and gradation for all subgrades, subbases and bases. From this study it appears that a correlation exists between "percent design" and life of pavements.

A report covering the development and use of triaxial tests for subgrade soils and flexible base materials was presented to the Highway Research Board by the author in 1946. In 1949, we submitted another report including a classification chart and a depth of pavement table which were published in Highway Research Board Bulletin 8-R. The classification chart has been revised once since then in 1952 so as to accommodate for the classification of cohesionless sands. The depth of pavement table was revised and presented in graphical form in 1951 to avoid having to distinguish between high and low modulus base materials.

THE testing procedure used today for disturbed soils is still essentially the same as reported in 1946, except that the use of some newer and better testing machines have been adopted. For the benefit of those who are not familiar with the particular method of testing used, the following is a brief summary of the method. A more detailed procedure, THD-80, is included in the appendix.

1. A 200- to 300-lb. sample is air dried and separated into various ranges of particle sizes. Portions retained on the 2-inch screen are crushed or excluded.

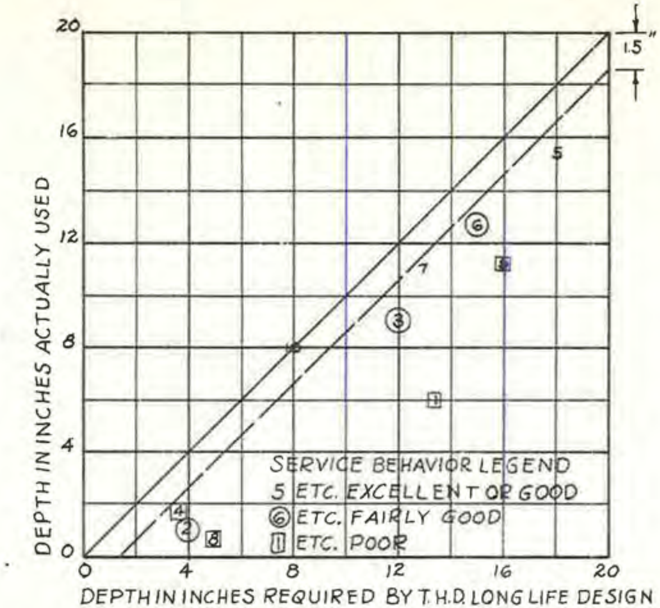
2. A moisture-density curve for a selected compactive effort is obtained by molding 6-inch-diameter by 8-inch-height specimens in four equal layers of 2-inch thickness each. The batch for each specimen consists of grain size components which have been recombined on the basis of the original gradation obtained in Step 1.

3. Six specimens, as nearly identical as possible, are compacted at optimum moisture content. All specimens are weighed, measured and extruded from the molds. Porous stones are placed on top and bottom.

4. After storing overnight, specimens consisting of materials that do not develop shrinkage cracks are partially dried by placing in an air-drying oven (forced draught at 140 F.) for a period of 8 hours. Upon removal, the specimens are allowed to stand overnight in the open laboratory. The specimens again are weighed. Usually, about two thirds to one half of the molding moisture is removed. Materials that tend to develop shrinkage cracks are not dried.

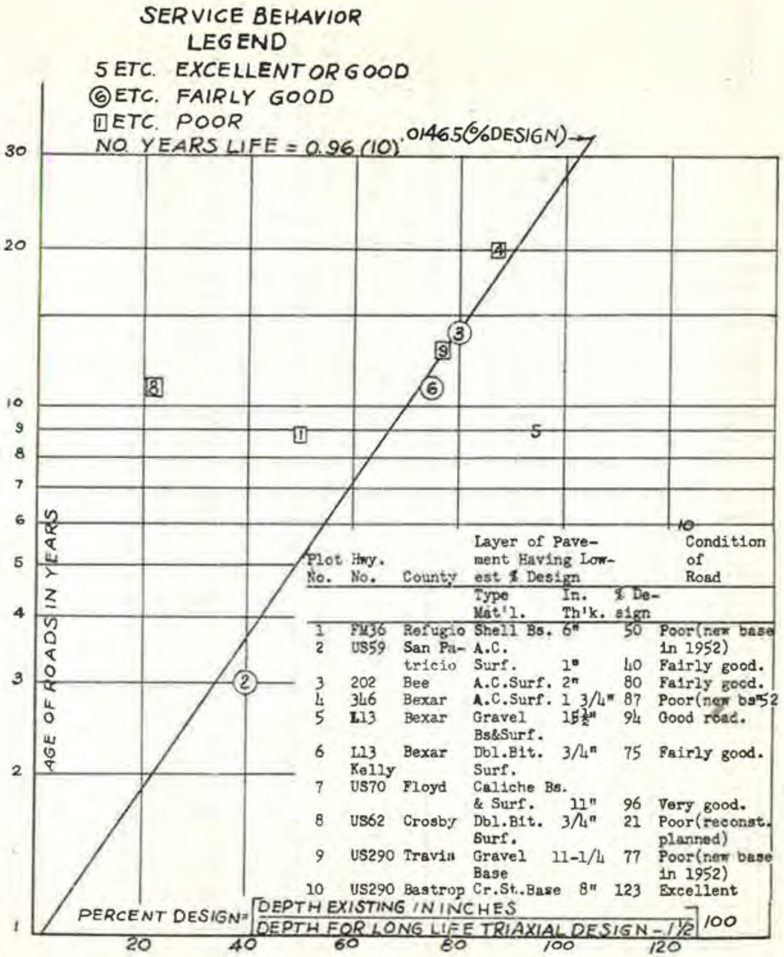
5. The axial cells, deflated by vacuum, are placed on the specimens. A suitable vertical surcharge (about 0.33 to 1.00 psi. for most subgrades) is placed on the top stone. The specimens are then placed in pans of water so that the water level is ½ inch below the bottom of the specimen itself. This assembly is then placed on the storage rack in the moisture room and connected with the constant pressure air manifold. The usual lateral pressure is 1 psi. The specimens are permitted to absorb water by capillarity until equilibrium is attained.

6. Each specimen is tested in compression at a constant lateral pressure. The six identical specimens are usually tested at lateral pressures of 0, 3, 5, 10, 15 and 20 psi., respectively. This pressure is applied by means of the cells, supplied by an auxiliary air tank. The rate of strain is 0.15 inch per minute. Simultaneous readings of load and defor-



Plot No.	Highway No.	County	Layer of Pavement Requiring Thickest Cover — Type Mat'l.	Inches Thickness — Existing	Inches Thickness — For Design	Condition of Road
1	FM-36	Refugio	Shell Base	6	13-1/2	Poor (new base in '52)
2	US-59	San Patricio	A.C. Surface	1	4	Fairly good.
3	202	Bee	Caliche Base	9	12	Fairly good.
4	346	Bexar	A.C. Surface	1-3/4	3-1/2	Poor (new base in '52)
5	Loop 13	Bexar	Gravel Base & Surface	15-1/2	18	Good road.
6	Loop 13 Kelly	Bexar	Dbl. Bit. Surf.	12-3/4	15	Fairly good.
7	US-70	Floyd	Caliche Base & Surface	11	13	Very good.
8	US-62	Crosby	Dbl. Bit.	3/4	5	Poor (reconstruction planned.)
9	US-290	Travis	Gravel	11-1/4	16	Poor (new base in '52)
10	US-290	Bastrop	Cr. St. Base	8	8	Excellent

Figure I. Relation of actual depths used to depths required by T.H.D. triaxial test for layers of pavement requiring greatest thickness of coverage.



Plot No.	Hwy. No.	County	Layer of Pavement Having Lowest % Design — Type Mat'l.	In. Th'k.	% Design	Condition of Road
1	FM36	Refugio	Shell Bs.	6"	50	Poor(new base in 1952)
2	US59	San Patricio	A.C. Surf.	1"	40	Fairly good.
3	202	Bee	A.C.Surf.	2"	80	Fairly good.
4	346	Bexar	A.C.Surf.	1 3/4"	87	Poor(new bs'52
5	L13	Bexar	Gravel Bs&Surf.	15½"	94	Good road.
6	L13 Kelly	Bexar	Dbl.Bit. Surf.	3/4"	75	Fairly good.
7	US70	Floyd	Caliche Bs. & Surf.	11"	96	Very good.
8	US62	Crosby	Dbl.Bit. Surf.	3/4"	21	Poor(reconst. planned)
9	US290	Travis	Gravel Base	11-1/4	77	Poor(new base in 1952)
10	US290	Bastrop	Cr.St.Base	8"	123	Excellent

Figure II. Relation of pavement life to percent design*.

TABLE I

Highway No.	County	Road Age Yrs.	Wheel Load* (in Pounds)	Pavement Layer Thickness (Inches)	Triaxial Strength Class	No. Inches Coverage Required by Tests**	Percent Design***	Service Behavior of Road
FM-136	Refugio	9	6,100	½" single asph. surf. 5½" shell Clay Subg.	 1 4.9	 1 13½	 **** 50	New base placed over old base in 1952
US-59	San Patricio	3	12,500	1" A.C. surfacing 10" caliche Subgrade	 3.1 3.4	 4 7	 40 200	Fairly good.
202	Bee	14	5,000	2" A.C. Surfacing 7" caliche Subgrade	 3.3 4.8	 4 12	 80 86	Fairly good.
346	Bexar	20	4,500	1¾" A.C. surfacing 13" gravel Subgrade	 3.1 3.1	 3½ 3½	 87 737	Poor; new base placed over old base in 1952
Loop 13 Lackland Air bs.	Bexar	9	8,200	2½" A.C. surfacing 13" gravel Subgrade	 2.8 5.2	 2¾ 18	 200 94	Good.
Loop 13 Kelly Overpass	Bexar		8,100	¾" dbl. bit. surf. 12" gravel Subgrade	 2.7 4.7	 2½ 15	 75 94	Fairly good.
US-70	Floyd	14	6,700	2" triple bit. surf. 9" caliche Subgrade	 3.1 4.6	 3½ 13	 100 96	Excellent
US-62	Crosby	11	7,900	¾" dbl. bit. surf. 7" caliche Subgrade	 3.4 4.4	 5 12	 21 74	Reconstruction planned.
US-290	Travis	13	8,400	1¼" triple bit. surf. 10" gravel Subgrade	 3.0 4.9	 3 16	 83 77	Reconstructed after 13 yrs. service.
US-290	Bastrop	6	16,000	¼" single bit. surf. 7¾" cr. stone base Subgrade	 1 3.4	 1 8	 **** 123	Excellent

*Estimated average of 10 heaviest wheel loads per av. day during years road has been in service. Data furnished by THD Highway Planning Survey Division.

**Triaxial tests for long-life design.

***Percent Design = (Depth existing divided by depth required by tests minus 1½ in.) 100

****Percent design is very high but cannot be determined from the formula.

mation are taken at intervals of 0.01 inch of deformation. Loading continues until the specimen fails.

7. After the completion of the compression test, the entire specimen is dried at 110 C. On the basis of the total dry weight, extra data as to density, moisture content, moisture absorption, etc., may be calculated.

8. From the principal stresses at the instant of failure, Mohr's diagram of stress is constructed.

9. A portion of the Mohr envelope is transferred to a classification chart and the strength class of the material is determined to the nearest tenth.

10. The depth of coverage in inches for

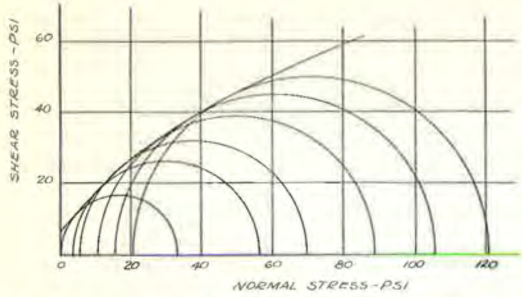


Figure 1. Mohr's Diagram, Lab. No. 50-72-R San Patricio County US 59.

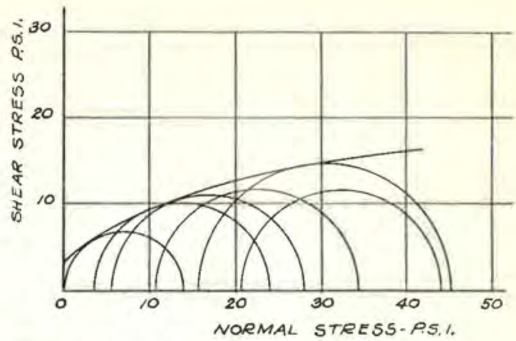


Figure 4. Mohr's Diagram, Lab. No. 50-81-R Bexar County, Loop 13, Kelly Field Overpass.

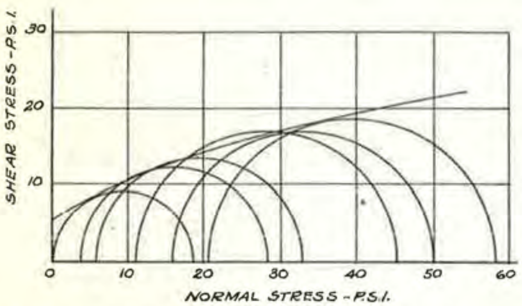


Figure 2. Mohr's Diagram, Lab. No. 50-93-R Crosby County, US 62.

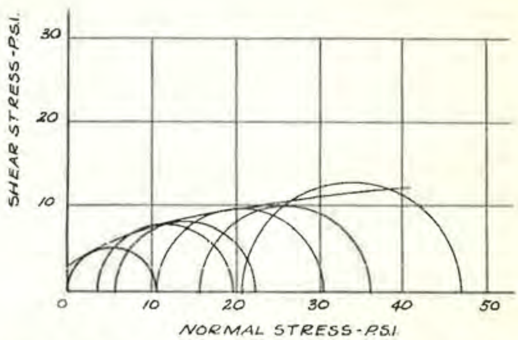


Figure 5. Mohr's Diagram, Lab. No. 50-77-R Refugio County F.M. 136.

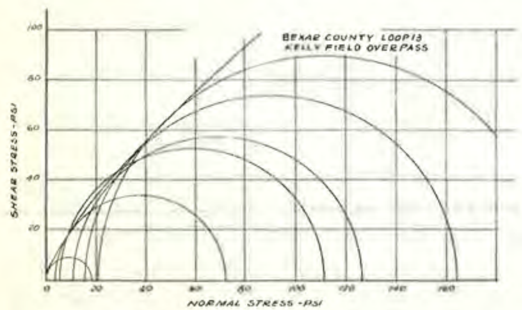


Figure 3. Mohr's Diagram, Lab. No. 50-80-R.

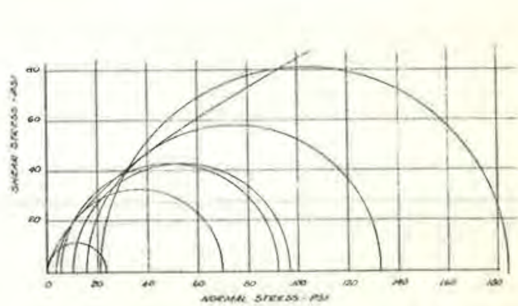


Figure 6. Mohr's Diagram, Lab. No. 50-78-R Bexar County, Loop 13, Lackland Air Force Base.

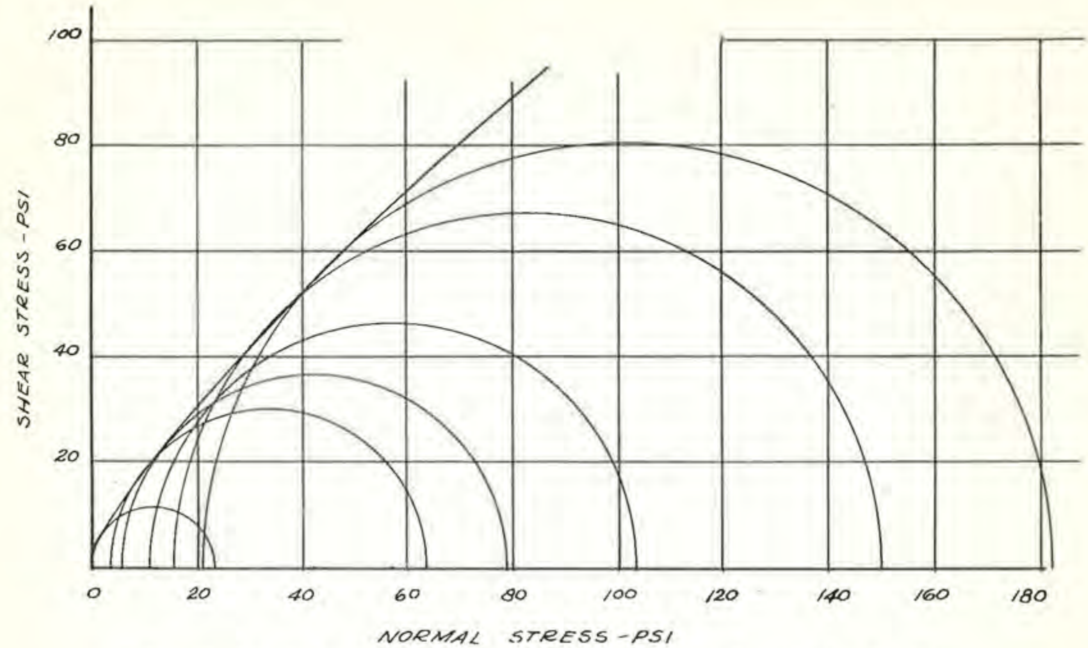


Figure 7. Mohr's Diagram, Lab.No.50-82-R, Bexar County St. Hwy.346.

a given wheel load is obtained by entering these data on a design chart.

The entire procedure when reduced to its simplest terms consists of comparing strength to stress for all layers of flexible pavement. The strength of the material is determined from carefully controlled triaxial tests and compared to a given wheel-load-stress condition by plotting the shear strength envelope on the above-mentioned classification chart. This chart has been derived from experience and

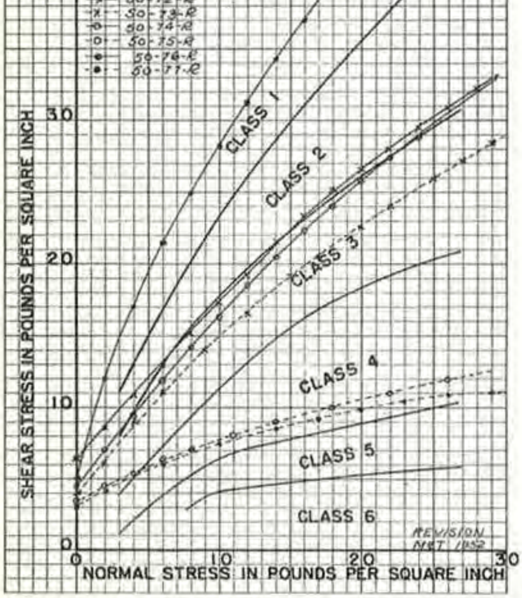


Figure 8.

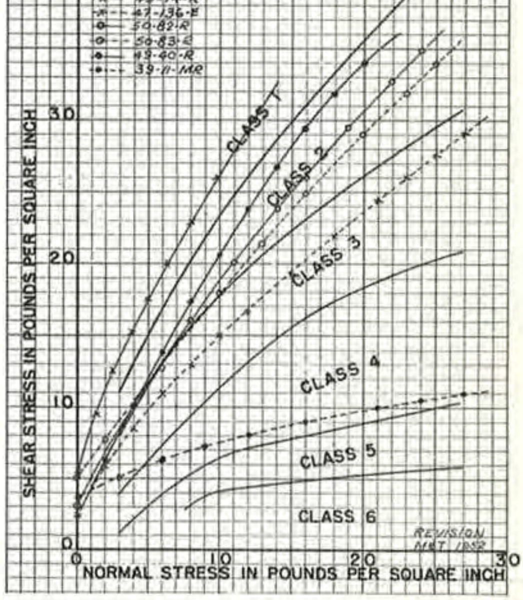


Figure 9.

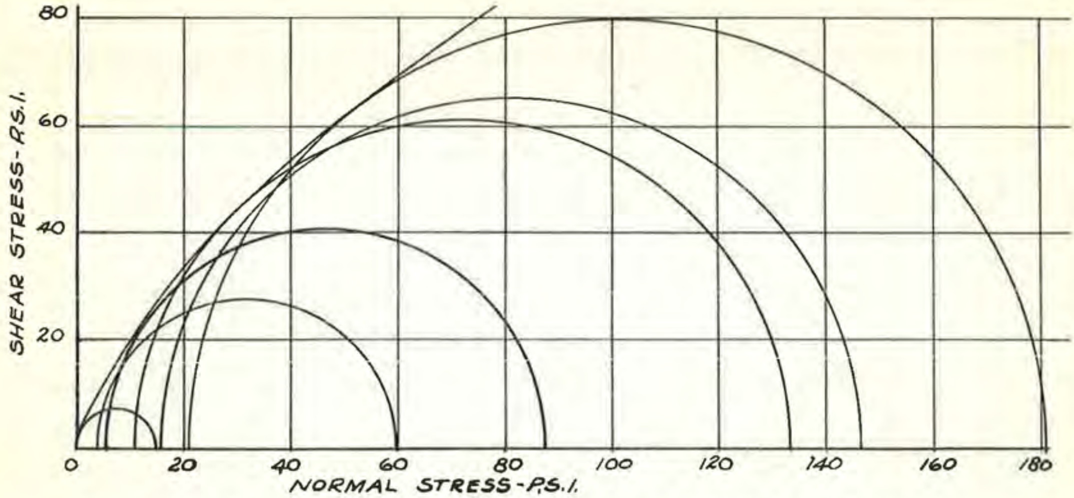


Figure 10. Mohr's Diagram, Lab.No.49-40-R, Travis County US 290.

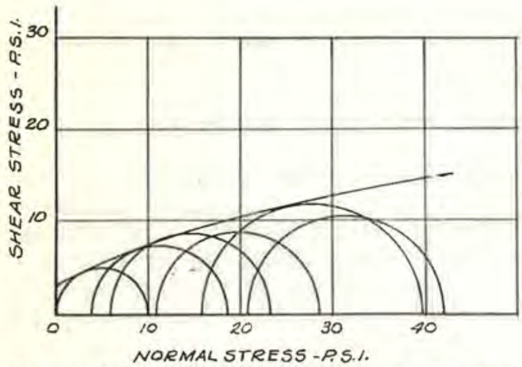


Figure 11. Mohr's Diagram, Lab. No. 50-75-R Bee County St. Hwy. 202.

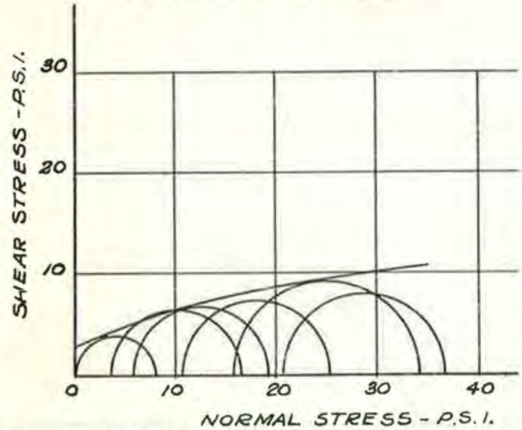


Figure 12. Mohr's Diagram, Lab. No. 50-84-R Bexar County, Loop 13, Lackland Air Force Base.

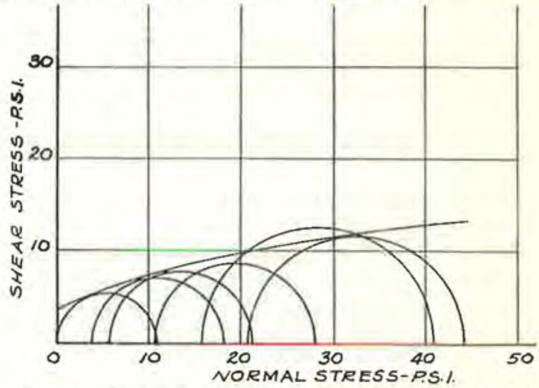


Figure 13. Mohr's Diagram, Lab. No.39-11-MR Travis County, US 290.

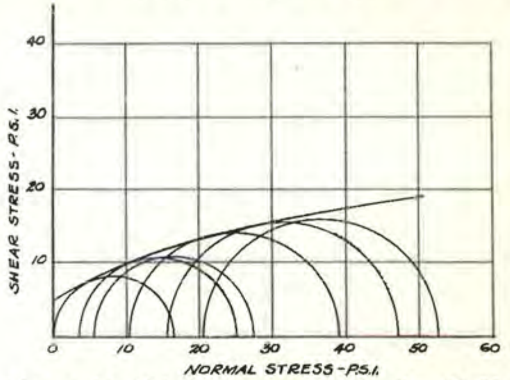


Figure 14. Mohr's Diagram, Lab. No. 50-91-R Floyd County, US 70.

certain theoretical concepts. The stress conditions for a variety of wheel loads, based upon constant tire pressures, may be analyzed by use of the pavement-analysis chart mentioned above. The primary purpose of this report is to show how well this method of analysis correlates with actual service behavior of flexible pavements. In order to do this, a thorough analysis of pavements, including service behavior, age, weight and volume of traffic, thicknesses of pavement layers, triaxial tests, soil constants and gradation on subgrades, subbases and bases, was made on ten projects located in south, central, and west Texas. Samples weighing 200 to 300 lb. each were taken from each layer of subgrade, subbase or base after cutting large holes in existing pavements. At this time thicknesses of all layers of pavements were measured. The age of the pavement and an evaluation of its service behavior were obtained from

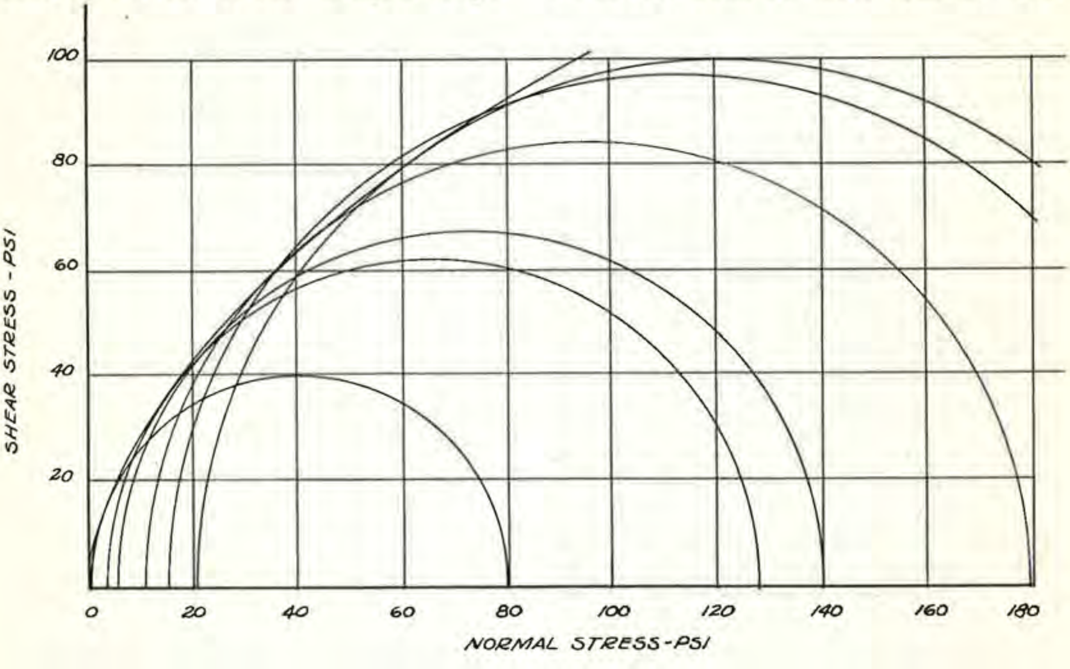


Figure 15. Mohr's Diagram, Lab. No. 50-76-R, Refugio County F.M. 136.

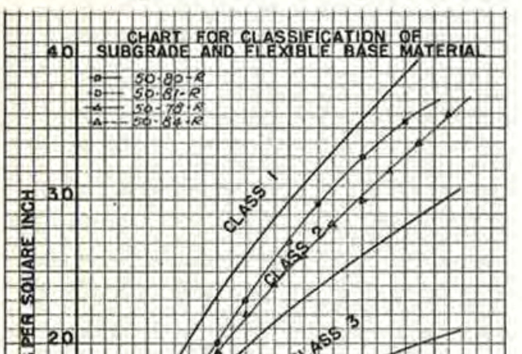


Figure 16.

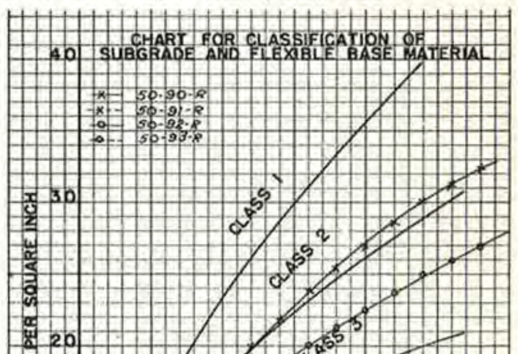


Figure 17.

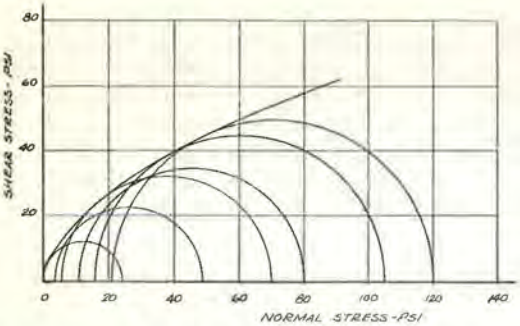


Figure 18. Mohr's Diagram, Lab. No. 50-90-R Floyd County, US 70.

state highway district and Bureau of Public Roads' personnel. Soil constants and gradation are shown on form sheets 476-A (see appendix). Results of triaxial tests for all soils tested are plotted in the form of Mohr diagrams as Figures 1-7, 10-15, and 18-24. Strength classifications for all soils tested are shown in Figures 8, 9, 16, and 17. Pertinent data pertaining to compaction, curing, absorption, and testing of all specimens are shown in Tables 1 to 20.

To study these projects objectively, estimates of the average of the ten-heaviest wheel loads per average day during the life of these pavements were obtained. Since these are averages over a period of years, it may be noted that many of the old projects showed much-lower wheel-load averages than newer projects.

Wheel-load data and test results are shown in Table I. Data shown in this tabulation, for layers of pavements needing greatest increase in thickness as judged by triaxial design, are plotted in Figure I, so as to show the relation of existing depths for design thicknesses. Figure I indicates that good long-life pavements may be approximately $1\frac{1}{2}$ inches thinner

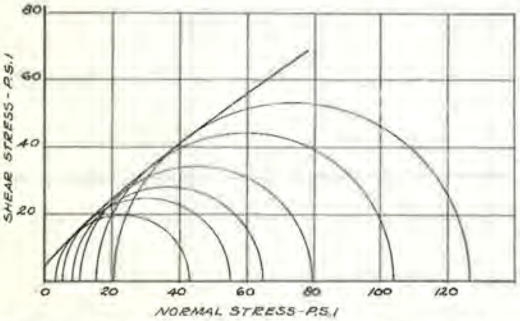


Figure 19. Mohr's Diagram, Lab. No. 47-136-E Bastrop County, US 290.

than required by our design procedure; therefore, in construction of new projects we often reserve an inch or two of surfacing for future application. It may also be noted that Points 2 and 4 are close together on this chart, although one is from a fairly good road the other was from a poor road, the difference being that the poor road is 20 years old and the fairly good road is only 3 years old. Figure II shows the relation of road age to "percent design" expressed as 100 (ratio of existing depth to depth required by triaxial tests minus $1\frac{1}{2}$ in.). This chart separates Points 2 and 4 more nearly as should be.

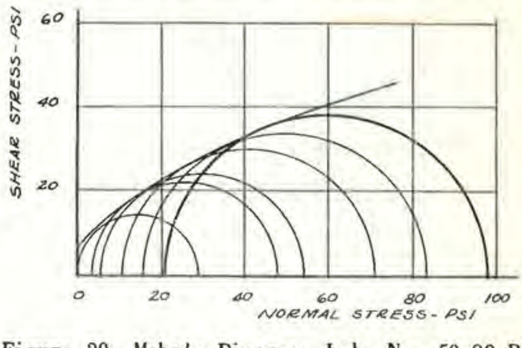


Figure 20. Mohr's Diagram, Lab. No. 50-92-R Crosby County, US 62.

In fact, all points tend to be arranged so that the line represented by the expression, "No. years life = . . . ," divides points for road failures on the left from points for good and excellent roads on the right of the line. The results shown are limited; however, for the time being they will help explain why some underdesigned pavements are not failing rapidly. It is also believed that these relations will help designers select design thicknesses in keeping with engineering and economical needs, whether for a short-life road requiring relocation soon, for stage construction, or for long-life urban sec-

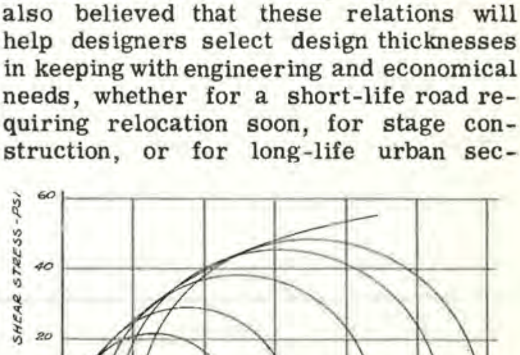


Figure 21. Mohr's Diagram, Lab. No. 50-74-R Bee County St. Hwy. 202.

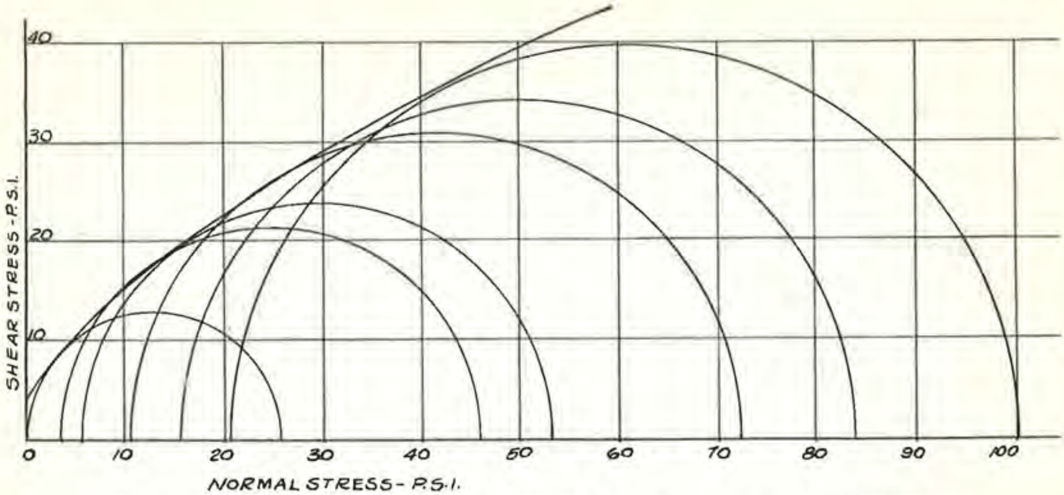


Figure 22. Mohr's Diagram, Lab. No. 50-73-R San Patricio County US 59.

tions. It is doubtful that the term "percent design" is of much value in estimating the life of bituminous surfacings which are less than $1\frac{1}{2}$ inches thick, because the triaxial method does not measure the properties of asphalts. Since the amount of maintenance required for roads is somewhat dependent upon skill and timing of operations, it is by no means certain that projects having similar percentages of design will require the same amounts of maintenance. General observation of some of these projects throughout their life bears out this statement.

For reconstruction of the above roads it is suggested that design loads should be increased considerably above those found to exist in the past because the weight of traffic in the future cannot usually be expected to be as light as it has been in the past. Planning Survey Divisions can be of much assistance in furnishing wheel load data for design. It is suggested that each layer in the entire pavement system be rechecked for percent design when proposing the application of new layers for reconstruction.

When designing for construction of new projects, the following steps are recommended:

1. Use USDA county soil maps and geological survey data to form a soils area concept or at least a soils reconnaissance before sampling for triaxial tests. If this is done the number of subgrade soil samples required per mile may be as low as two or three, and the data may be applicable to other areas

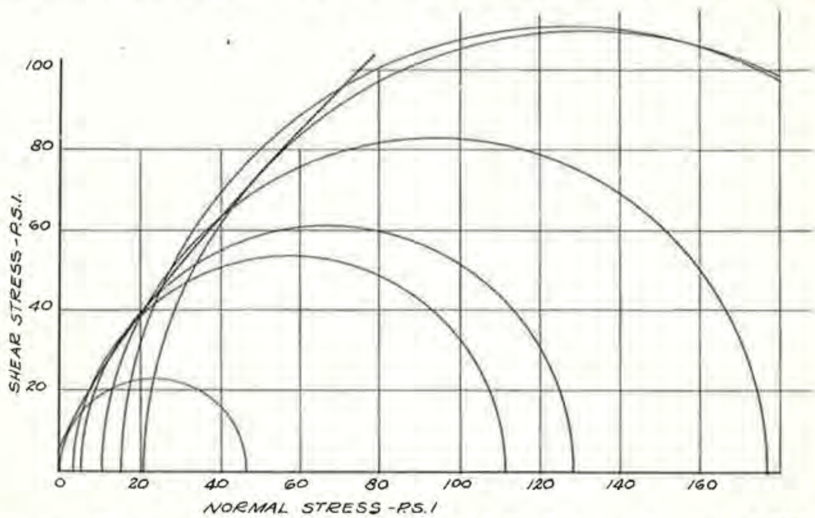


Figure 23. Mohr's Diagram, Lab. No. 49-14-R, Bastrop County US 290.

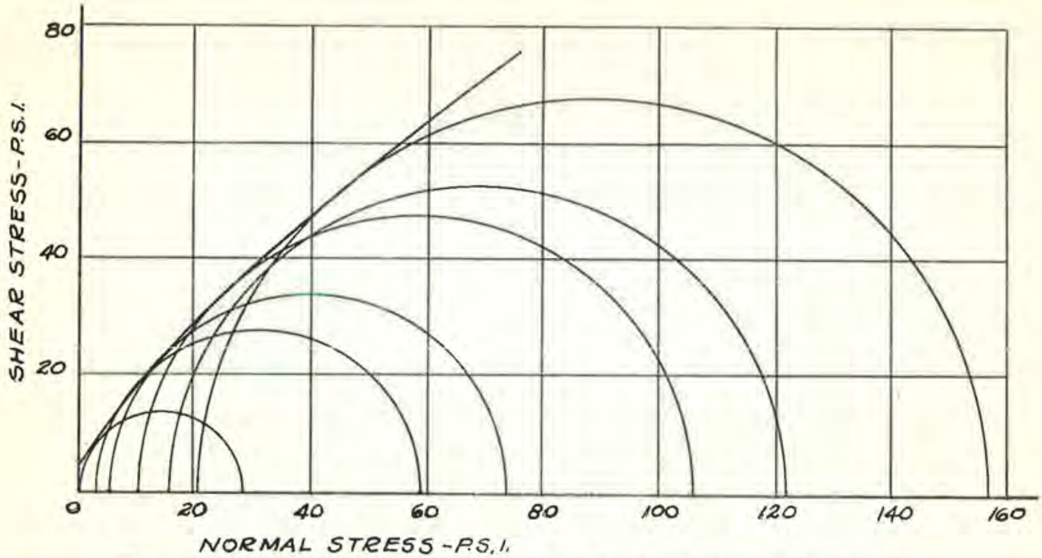


Figure 24. Mohr's Diagram, Lab.No. 50-83-R, Bexar County St. Hwy. 346.

shown by the map to be from a similar soil series and to have similar physical constants.

2. Selection of percent design depends upon the life of the road desired, the ease with which new layers of base or surfacing can be added in the future, and certain economical considerations.

3. It should be realized that designing for strength alone does not prevent the occurrence of detrimental shrinkage cracks. To avoid these it is suggested that granular layers or asphalt membranes be extended as far outside the edges of pavement as is economically feasible. This type of construction is essential only in high volume change areas or where the soil forms extensive shrinkage cracks.

4. In order to be assured of securing strengths of subgrades and pavement layers comparable to those obtained in testing, a method of compaction control is recommended. The method is fully described in a separate paper at this meeting by the author, entitled: "Selection of Densities for Subgrades and Flexible Base Materials."

CONCLUSIONS

1. Since the data obtained from triaxial tests made it possible to correlate percent design with life of pavements, it appears that triaxial methods can be used successfully to analyze most highway flexible pavement problems. In addition to estimating overall thicknesses on subgrades, the method also determines the quality and density requirements of subbase and flexible base materials, the latter being equally, if not more, important than overall thickness because good pavements are a prerequisite to proper thickness evaluations. The term "percent design" appears to be a basic expression and is necessary in order to have a good understanding of flexible-pavement design.

2. The simplicity and workability of the method are attested to by the fact that approximately half of the district offices either have been or are being equipped for triaxial testing. This program has been carried out voluntarily by the Districts, and is not mandatory.

ACKNOWLEDGEMENT

The writer is indebted to many who have contributed, encouraged or assisted in the development of triaxial testing for use in the design of pavements. A few of these people or organizations have been mentioned in the text; however, it should be mentioned that the work of the members of the Soils Section and other members of the Materials and Tests Division of the Texas Highway Department has been a major factor in making this report possible. The author is also indebted for assistance received from personnel of the Ft. Worth office of Bureau of Public Roads and the Road Design and Highway-Planning Survey divisions of the Texas Highway Department.

THD-80

Table No. 1

Lab. No. 50-72-R County San Patricio Hwy. US 59 Project

Material Caliche Base Identification

Description 10 inches of caliche

Opt. Moist. 14.8 Opt. Dry Density 113.3 at Comp. Effort 13.26 ft.lb./cu.in.

Spec. No.	Molding Data: % Moist. Dry Weight	Molding Data: Dry Den. Lbs./Cu.Ft.	Curing Data Moist. Cont.: % After Dry.	Curing Data Moist. Cont.: % Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations, Straight Portion, From: PSI Unit Stress	From: % Strain	To: PSI Unit Stress	To: % Strain	Ultimate: PSI Unit Stress	Ultimate: % Strain
4	14.8	114.1	9.8	16.5	Small	0.0	14.0	0.80	26	1.3	33.3	1.91
5	14.9	112.9	9.7	17.2	"	3.6	21	0.9	38	1.5	56.2	2.77
6	15.0	113.5	9.8	17.0	"	5.6	19	0.6	45.5	1.5	69.4	3.93
7	15.1	113.2	9.6	17.2	"	10.6	30	0.9	54	1.55	88.4	4.81
8	15.0	113.0	9.8	17.3	"	15.6	38	0.9	67	1.55	105.1	4.66
12	14.0	112.9	9.4	17.7	"	20.6	45	0.9	85	1.6	120.3	4.52

Remarks

THD-80

Table No. 3

Lab. No. 50-74-R County Bee Hwy. 202 Project

Material Caliche Base Identification

Description 7" of Caliche Base

Opt. Moist. 17.7 Opt. Dry Density 107.8 at Comp. Effort 13.26 ft.lb./cu.in.

Spec. No.	Molding Data: % Moist. Dry Weight	Molding Data: Dry Den. Lbs./Cu.Ft.	Curing Data Moist. Cont.: % After Dry.	Curing Data Moist. Cont.: % Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations, Straight Portion, From: PSI Unit Stress	From: % Strain	To: PSI Unit Stress	To: % Strain	Ultimate: PSI Unit Stress	Ultimate: % Strain
3	17.6	107.9	17.6	21.5	Small	0.0	14.0	0.8	21.0	1.4	23.3	1.78
9	17.6	108.1	18.3	21.5	"	3.7	22	1.2	38	2.2	47.2	3.30
6	17.7	108.1	17.8	21.6	"	5.7	28	1.15	46	2.1	64.4	4.32
5	17.8	108.1	17.7	21.7	"	10.7	26	0.75	52	1.9	87.9	5.98
8	17.8	107.2	17.7	22.0	"	15.7	30	0.7	56	1.7	106.3	6.06
7	17.8	107.1	17.7	22.1	"	20.7	31	0.6	60	1.65	119.7	7.56

Remarks

THD-80

Table No. 2

Lab. No. 50-73-R County Hwy. US 59 Project

Material Subgrade Soil Identification

Description

Opt. Moist. 11.6 Opt. Dry Density 118.7 at Comp. Effort 13.26 ft.lb./cu.in.

Spec. No.	Molding Data: % Moist. Dry Weight	Molding Data: Dry Den. Lbs./Cu.Ft.	Curing Data Moist. Cont.: % After Dry.	Curing Data Moist. Cont.: % Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations, Straight Portion, From: PSI Unit Stress	From: % Strain	To: PSI Unit Stress	To: % Strain	Ultimate: Stress	Ultimate: % Strain
5	11.3	118.8	9.8	14.0	Small	0	11.5	0.60	18.0	1.25	25.9	2.5
6	11.6	118.6	10.1	14.0	"	3.6	14.0	0.50	26.0	1.40	46.0	5.0
4	11.9	118.5	9.6	13.9	"	5.6	18.5	0.50	26.0	0.90	53.4	5.0
8	11.7	118.7	10.1	13.8	"	10.6	18.0	0.40	39.0	1.10	72.1	5.9
7	11.7	118.8	10.1	13.9	"	15.6	18.0	0.40	50.0	1.50	83.9	7.4
9	11.7	119.0	10.3	13.8	"	20.6	26.0	0.50	54.0	1.40	100.1	7.1

Remarks

THD-80

Table No. 4

Lab. No. 50-75-R County Hwy. 202 Project

Material Subgrade Soil Identification

Description

Opt. Moist. 16.8 Opt. Dry Density 107.0 at Comp. Effort 6.63 ft.lb./cu.in.

Spec. No.	Molding Data: % Moist. Dry Weight	Molding Data: Dry Den. Lbs./Cu.Ft.	Curing Data Moist. Cont.: % After Dry.	Curing Data Moist. Cont.: % Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations, Straight Portion, From: PSI Unit Stress	From: % Strain	To: PSI Unit Stress	To: % Strain	Ultimate: PSI Unit Stress	Ultimate: % Strain
6	16.8	106.1	16.0	20.4	1.8	0	2.5	0.10	5.5	0.30	9.8	1.7
7	16.4	107.1	15.3	20.1	3.0	3.6	5.0	0.15	9.5	0.30	18.5	2.3
2	17.9	106.4	15.6	20.1	1.5	5.6	6.0	0.10	12.5	0.40	22.9	3.0
5	16.6	107.3	15.8	20.0	2.8	10.6	8.0	0.15	14.5	0.40	28.4	2.8
10	16.6	107.4	16.1	19.8	2.2	15.6	12.5	0.15	20.0	0.40	39.4	3.5
9	16.7	107.4	16.0	20.0	3.0	20.6	14.5	0.10	21.0	0.40	41.7	3.9

Remarks

THD-80

Table No. 5

Lab. No. 50-76-R County Refugio Hwy. FM-136 Project

Material Shell Identification

Description 5-1/2 inches of shell base

Opt. Moist. 12.5 Opt. Dry Density 119.6 at Comp. Effort 13.26 ft.lb./cu.in.

	Molding Data		Curing Data Moist. Cont.				Stress-Strain Relations					
							Straight Portion From		Straight Portion To		Ultimate	
Spec. No.	% Moist. Dry Weight	Dry Den. Lbs./Cu.Ft.	% After Dry.	% Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain
9	12.4	119.5	5.0	10.6	Small	0.0	22	1.05	50	2.25	79.9	5.25
11	12.4	119.5	4.7	10.5	"	3.6	30	0.9	83	2.75	127.3	6.31
12	12.5	119.4	5.1	10.9	"	5.6	42	1.2	83	2.40	139.7	6.18
7	12.3	119.7	5.2	10.7	"	10.6	43	0.8	92	1.85	187.8	7.51
8	12.4	119.6	5.1	10.7	"	15.6	28	0.4	83	1.60	208.5	7.75
10	12.7	119.6	5.2	10.7	"	20.6	20	0.2	85	1.40	220.0	7.12

Remarks

THD-80

Table No. 6

Lab. No. 50-77-R County Hwy. FM-136 Project

Material Subgrade Soil Identification

Description

Opt. Moist. 17.9 Opt. Dry Density 104.7 at Comp. Effort 6.63 ft.lb./cu.in.

	Molding Data		Curing Data Moist. Cont.				Stress-Strain Relations					
							Straight Portion From		Straight Portion To		Ultimate	
Spec. No.	% Moist. Dry Weight	Dry Den. Lbs./Cu.Ft.	% After Dry.	% Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain
2	17.9	104.1	15.4	21.2	1.8	0	0.0	0.0	6.0	0.30	10.5	1.2
6	17.9	104.6	18.3	21.3	2.7	3.6	1.0	0.0	11.0	0.40	19.7	2.6
4	18.0	104.6	15.8	21.4	2.4	5.6	2.0	0.0	12.0	0.40	22.3	2.8
7	17.7	104.9	17.4	21.7	3.6	10.6	6.5	0.1	19.0	0.65	30.3	3.1
5	17.8	105.0	18.0	21.3	2.5	15.6	7.0	0.0	22.5	0.60	35.9	2.7
8	18.0	105.2	17.8	21.4	3.0	20.6	10.5	0.0	34.0	0.75	46.9	2.8

Remarks

THD-80

Table No. 7

Lab. No. 50-78-R County Bexar Hwy. Loop 13 Project

Material Gravel Identification

Description 13" Gravel Base near Lackland Air Base

Opt. Moist. 6.0 Opt. Dry Density 135.4 at Comp. Effort 13.26 ft.lb./cu.in.

	Molding Data		Curing Data Moist. Cont.				Stress-Strain Relations					
							Straight Portion From		Straight Portion To		Ultimate	
Spec. No.	% Moist. Dry Weight	Dry Den. Lbs./Cu.Ft.	% After Dry.	% Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain
4	6.8	135.0	4.0	8.2	Small	0.0	0.0	0.0	9.0	0.5	22.6	2.82
6	6.8	135.4	3.9	7.9	"	3.6	15.0	0.5	28.0	0.9	69.8	3.13
1	6.6	135.8	3.7	8.1	"	5.6	23.0	0.6	46.0	1.05	91.3	4.63
4	6.8	135.9	3.9	8.0	"	10.6	28.0	0.6	67.0	1.45	96.2	4.13
7	6.7	136.4	3.9	7.7	"	15.6	39.0	0.75	68.0	1.25	132.5	7.06
5	6.8	136.3	3.9	7.8	"	20.6	40.0	0.55	100.0	1.35	183.8	4.92

Remarks

THD-80

Table No. 8

Lab. No. 50-84-R County Hwy. Loop 13 Project

Material Subgrade Soil Identification

Description Subgrade near Lackland Air Base

Opt. Moist. 33.4 Opt. Dry Density 80.7 at Comp. Effort 4.0 ft.lb./cu.in.

	Molding Data		Curing Data Moist. Cont.				Stress-Strain Relations					
							Straight Portion From		Straight Portion To		Ultimate	
Spec. No.	% Moist. Dry Weight	Dry Den. Lbs./Cu.Ft.	% After Dry.	% Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain
2	33.9	79.7	36.6	40.5	3.18	0	0.5	0.0	5.5	0.40	7.9	1.02
4	33.3	80.0	38.0	40.5	4.69	3.6	1.5	0.0	9.5	0.35	16.6	1.78
7	33.3	80.9	36.1	40.4	5.24	5.6	2.0	0.0	12.5	0.50	18.9	1.91
6	33.2	81.1	36.0	39.5	5.87	10.6	6.0	0.15	15.0	0.65	25.1	2.54
5	33.4	81.2	35.3	40.2	5.23	15.6	7.0	0.0	23.0	0.80	33.9	3.84
8	33.5	81.2	36.4	40.5	5.30	20.6	13.5	0.15	20.0	1.00	36.4	2.82

Remarks

THD-80

Table No. 9

Lab. No. 50-80-R County Bexar Hwy. Loop 13 Project

Material 12" Gravel Base Identification

Description Near Kelly Field Overpass

Opt. Moist. 6.2 Opt. Dry Density 139.4 at Comp. Effort 13.26 ft.lb./cu.in.

Spec. No.	Molding Data: % Moist. Dry Weight	Molding Data: Dry Den. Lbs./Cu.Ft.	Curing Data Moist. Cont.: % After Dry.	Curing Data Moist. Cont.: % Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations, Straight Portion, From: PSI Unit Stress	From: % Strain	To: PSI Unit Stress	To: % Strain	Ultimate: PSI Unit Stress	Ultimate: % Strain
9	6.2	139.1	3.4	6.4	Small	0.0	0.0	0.0	11.0	0.9	18.5	2.07
10	6.3	138.6	3.6	6.8	"	3.6	22.0	0.9	47.0	1.65	72.2	4.25
11	6.2	139.4	3.1	6.4	"	5.6	26.0	0.9	66.0	2.1	110.9	5.97
12	6.1	139.5	3.3	6.2	"	10.6	27.0	0.65	74.0	1.9	125.6	5.84
13	6.1	139.0	3.2	6.3	"	15.6	28.0	0.55	100.0	2.30	163.7	6.32
14	6.1	139.0	3.3	6.5	"	20.6	30.0	0.30	102.0	1.65	200.6	6.21

Remarks

THD-80

Table No. 10

Lab. No. 50-81-R County Hwy. Loop 13 Project

Material Subgrade Soil Identification

Description Near Kelly Field Overpass

Opt. Moist. 24.8 Opt. Dry Density 92.8 at Comp. Effort 6.63 ft.lb./cu.in.

Spec. No.	Molding Data: % Moist. Dry Weight	Molding Data: Dry Den. Lbs./Cu.Ft.	Curing Data Moist. Cont.: % After Dry.	Curing Data Moist. Cont.: % Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations, Straight Portion, From: PSI Unit Stress	From: % Strain	To: PSI Unit Stress	To: % Strain	Ultimate: PSI Unit Stress	Ultimate: % Strain
11	23.5	93.7	25.7	31.3	5.0	0	0.5	0.0	8.0	0.40	14.1	1.37
10	25.2	92.4	25.0	30.8	4.5	3.6	5.0	0.15	14.0	0.55	23.8	2.73
12	25.0	92.6	25.1	30.2	4.9	5.6	8.0	0.21	17.0	0.60	27.9	2.62
5	25.5	92.3	23.7	30.8	5.0	10.6	4.5	0.0	21.0	0.80	34.1	4.11
8	24.9	92.8	24.1	30.3	4.4	15.6	16.0	0.35	24.0	0.70	44.9	4.35
9	24.9	93.1	24.4	30.5	5.2	20.6	10.0	0.0	26.0	0.75	43.5	3.70

Remarks

THD-80

Table No. 11

Lab. No. 50-82-R County Bexar Hwy. 346 Project

Material 13" Gravel Base Identification

Description

Opt. Moist. 5.8 Opt. Dry Density 138.9 at Comp. Effort 13.26 ft.lb./cu.in.

Spec. No.	Molding Data: % Moist. Dry Weight	Molding Data: Dry Den. Lbs./Cu.Ft.	Curing Data Moist. Cont.: % After Dry.	Curing Data Moist. Cont.: % Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations, Straight Portion, From: PSI Unit Stress	From: % Strain	To: PSI Unit Stress	To: % Strain	Ultimate: PSI Unit Stress	Ultimate: % Strain
2	5.7	138.4	5.7	6.2	Small	0.0	6.0	0.35	14.0	1.0	21.7	1.21
7	5.8	138.7	5.5	6.3	"	3.6	14.0	0.50	34.0	1.30	63.5	4.75
5	5.8	138.8	5.4	6.4	"	5.6	12.0	0.30	33.0	1.25	78.8	4.50
8	5.8	138.9	5.4	6.3	"	10.6	26.0	0.65	60.0	1.95	103.5	4.51
4	5.8	139.1	5.7	6.2	"	15.6	36.0	0.65	88.0	2.2	149.5	5.80
6	5.8	139.6	5.3	6.1	"	20.6	54.0	1.05	100.0	2.15	181.9	6.07

Remarks

THD-80

Table No. 12

Lab. No. 50-83-R County Hwy. 346 Project

Material Subgrade Soil Identification

Description

Opt. Moist. 10.2 Opt. Dry Density 120.3 at Comp. Effort 13.26 ft.lb./cu.in.

Spec. No.	Molding Data: % Moist. Dry Weight	Molding Data: Dry Den. Lbs./Cu.Ft.	Curing Data Moist. Cont.: % After Dry.	Curing Data Moist. Cont.: % Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations, Straight Portion, From: PSI Unit Stress	From: % Strain	To: PSI Unit Stress	To: % Strain	Ultimate: PSI Unit Stress	Ultimate: % Strain
2	10.2	120.1	3.6	10.6	Small	0	10.0	0.50	17.5	0.90	28.0	2.19
5	10.2	120.2	3.4	10.7	"	3.6	12.0	0.25	31.0	0.90	59.4	2.79
6	10.3	120.2	3.6	10.6	"	5.6	12.0	0.25	35.5	0.90	73.5	3.17
8	10.3	120.3	3.5	10.5	"	10.6	22.0	0.40	46.0	0.80	106.0	3.56
4	10.1	120.4	3.3	10.8	"	15.6	23.0	0.35	62.0	0.95	121.6	3.67
7	10.2	120.4	3.5	10.5	"	20.6	28.0	0.30	77.0	0.95	156.0	3.56

Remarks

THD-80

Table No. 13

Lab. No. 50-90-R County Floyd Hwy. U.S. 70 Project

Material 9" Caliche Base Identification Station 330 + 15

Description

Opt. Moist. 11.3 Opt. Dry Density 120.4 at Comp. Effort 13.26 ft.lb./cu.in.

Spec. No.	Molding Data: % Moist. Dry Weight	Molding Data: Dry Den. Lbs./Cu.Ft.	Curing Data Moist. Cont.: % After Dry.	Curing Data Moist. Cont.: % Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations, Straight Portion, From: PSI Unit Stress	From: % Strain	To: PSI Unit Stress	To: % Strain	Ultimate: PSI Unit Stress	Ultimate: % Strain
9	11.3	120.1	7.7	13.2	Small	0.0	9.0	0.35	18.0	0.9	24.0	1.53
5	11.2	120.3	9.0	13.5	"	3.6	16.0	0.5	29.0	1.05	48.2	3.31
6	11.3	120.1	7.7	11.8	"	5.6	22.0	0.5	48.0	1.10	69.5	2.78
8	11.2	120.7	7.5	13.1	"	10.6	26.0	0.6	52.0	1.50	79.5	6.03
7	11.3	120.8	7.6	12.9	"	15.6	30.0	0.6	54.0	1.20	104.4	6.66
3	11.6	120.6	7.7	13.1	"	20.6	24.0	0.4	62.0	1.30	119.8	8.19

Remarks

THD 80

Table No. 14

Lab. No. 50-91-R County Floyd Hwy. U.S. 70 Project

Material Subgrade Identification

Description Clay Soil - Subgrade sampled from R.O.W. 10 miles East of Plainview. Station 330 + 15

Opt. Moist. 18.9 Opt. Dry Density 104.6 at Comp. Effort 6.63 ft.lb./cu.in.

Spec. No.	Molding Data: % Moist. Dry Weight	Molding Data: Dry Den. Lbs./Cu.Ft.	Curing Data Moist. Cont.: % After Dry.	Curing Data Moist. Cont.: % Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations, Straight Portion, From: PSI Unit Stress	From: % Strain	To: PSI Unit Stress	To: % Strain	Ultimate: PSI Unit Stress	Ultimate: % Strain
4	18.7	104.5	18.7	21.8	2.87	0.0	0.5	0.0	9.5	0.25	16.5	1.15
8	19.2	104.3	19.3	22.1	2.74	3.8	1.5	0.0	12.5	0.40	24.7	3.19
7	19.1	104.2	19.0	22.3	2.78	5.8	2.0	0.0	14.0	0.40	27.1	2.80
5	18.7	104.8	18.4	21.7	2.89	10.8	4.2	0.0	21.0	0.48	38.5	3.96
2	18.7	104.8	18.7	21.4	2.55	15.8	6.8	0.0	27.0	0.60	46.6	4.36
6	18.9	105.4	18.7	21.5	2.81	20.8	10.1	0.0	33.0	0.70	52.2	4.64

Remarks

THD-80

Table No. 15

Lab. No. 50-92-R County Crosby Hwy. U.S. 62 Project

Material 7" Caliche Base Identification

Description 1 Mile South Floyd-Crosby County Line

Opt. Moist. 12.6 Opt. Dry Density 116.0 at Comp. Effort 13.26 ft.lb./cu.in.

Spec. No.	Molding Data: % Moist. Dry Weight	Molding Data: Dry Den. Lbs./Cu.Ft.	Curing Data Moist. Cont.: % After Dry.	Curing Data Moist. Cont.: % Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations, Straight Portion, From: PSI Unit Stress	From: % Strain	To: PSI Unit Stress	To: % Strain	Ultimate: PSI Unit Stress	Ultimate: % Strain
9	12.6	115.6	11.5	16.2	Small	0.0	16.0	0.80	26.0	1.55	28.7	2.23
7	12.5	115.9	11.6	15.9	"	3.6	19.0	0.6	30.0	1.4	47.7	4.10
5	12.6	115.8	11.9	16.0	"	5.6	19.0	0.6	31.0	1.2	54.2	4.58
10	12.6	115.8	11.7	16.2	"	10.6	25.0	0.6	41.0	1.25	70.9	5.21
6	12.5	116.2	11.7	16.4	"	15.6	14.0	0.2	44.0	1.15	83.0	5.72
8	12.5	116.7	11.8	15.8	"	20.6	16.0	0.2	56.0	1.80	98.0	5.75

Remarks

THD-80

Table No. 16.

Lab. No. 50-93-R County Crosby Hwy. U.S. 62 Project

Material Subgrade Soil Identification

Description Sampled from R.O.W. one mile South Floyd-Crosby County Line

Opt. Moist. 14.9 Opt. Dry Density 112.3 at Comp. Effort 13.26 ft.lb./cu.in.

Spec. No.	Molding Data: % Moist. Dry Weight	Molding Data: Dry Den. Lbs./Cu.Ft.	Curing Data Moist. Cont.: % After Dry.	Curing Data Moist. Cont.: % Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations, Straight Portion, From: PSI Unit Stress	From: % Strain	To: PSI Unit Stress	To: % Strain	Ultimate: PSI Unit Stress	Ultimate: % Strain
1	14.5	113.9	15.7	17.3	3.78	0	0.0	0.0	7.0	0.30	18.8	2.35
4	15.0	112.5	14.4	18.1	3.15	3.7	1.5	0.0	12.0	0.40	28.2	3.96
2	15.9	112.4	16.5	18.0	5.29	5.6	2.2	0.0	13.0	0.45	36.6	5.46
6	15.5	112.1	14.8	18.5	2.40	10.6	4.5	0.0	18.0	0.50	46.1	6.45
7	14.6	112.6	14.1	17.9	2.44	15.6	7.0	0.0	26.0	0.75	49.8	5.58
8	15.0	112.2	14.6	18.4	4.00	20.7	11.0	0.0	34.5	0.85	55.6	4.08

Remarks

THD-80

Table No. 17

Lab. No. 49-40-R County Travis Hwy. U.S. 290 Project

Material 10" Gravel Base Identification

Description Sampled from R.O.W. East of Manor

Opt. Moist. 5.7 Opt. Dry Density 135.8 at Comp. Effort 13.26 ft.lb./cu.in.

Spec. No.	Molding Data		Curing Data Moist. Cont.				Stress-Strain Relations					
							Straight Portion				Ultimate	
							From		To			
	% Moist. Dry Weight	Dry Den. Lbs./Cu.Ft.	% After Dry.	% Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain
4	5.6	135.8	2.7	6.2	Small	0	5.0	0.25	12.0	0.70	14.6	1.16
5	5.6	135.3	2.9	6.2	"	3.6	14.0	0.30	42.5	0.80	59.5	1.93
6	5.7	135.0	3.0	6.2	"	5.6	14.0	0.20	54.0	0.80	86.8	2.06
7	5.7	136.4	3.1	5.9	"	10.6	12.0	0.10	106.0	1.40	133.0	2.60
8	5.6	135.4	2.8	6.2	"	15.6	40.0	0.30	99.5	0.75	145.9	2.06
10	5.7	136.0	3.0	6.1	"	20.6	40.0	0.30	121.0	0.95	179.8	2.98

Remarks

THD-80

Table No. 19

Lab. No. 49-44-R County Bastrop Hwy. 290 Project

Material 8" Crushed Stone Identification

Description Crushed Stone from Onion Creek

Pit Near Austin

Opt. Moist. 10.0 Opt. Dry Density 124.8 at Comp. Effort 13.26 ft.lb./cu.in.

Spec. No.	Molding Data		Curing Data Moist. Cont.				Stress-Strain Relations					
							Straight Portion				Ultimate	
							From		To			
	% Moist. Dry Weight	Dry Den. Lbs./Cu.Ft.	% After Dry.	% Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain
5	10.3	124.6	3.6	9.5	Small	0.0	7.5	0.0	23.0	0.60	47.0	1.95
6	10.3	124.8	2.9	9.5	"	3.6	35.0	0.45	75.0	1.40	110.8	2.99
3	9.3	124.0	2.6	9.6	"	5.6	37.5	0.40	83.0	1.40	147.1	3.54
7	10.1	125.0	2.9	9.4	"	10.6	10.0	0.0	135.0	1.55	176.8	3.25
8	10.2	124.9	3.4	9.6	"	15.6	15.0	0.0	137.5	1.20	236.5	3.51
9	10.1	125.2	2.9	9.2	"	20.6	15.0	0.0	157.5	1.40	239.9	3.39

Remarks

THD-80

Table No. 18

Lab. No. 39-11-MS County Travis Hwy. U.S. 290 Project

Material Identification

Description Sampled from R.O.W. East of Manor

Opt. Moist. 27.1 Opt. Dry Density 90.0 at Comp. Effort 5.25 ft.lb./cu.in.

Spec. No.	Molding Data		Curing Data Moist. Cont.				Stress-Strain Relations					
							Straight Portion				Ultimate	
							From		To			
	% Moist. Dry Weight	Dry Den. Lbs./Cu.Ft.	% After Dry.	% Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain
55	26.8	90.0	26.7	34.5	8.2	3.9	2.5	0.0	11.0	0.35	18.1	1.7
56	27.1	90.3	26.8	34.6	7.6	5.9	3.5	0.0	12.0	0.40	21.5	2.4
57	27.1	90.0	27.1	34.7	7.7	10.9	5.5	0.0	13.0	0.35	28.1	2.8
58	27.3	89.7	27.1	34.6	6.7	15.9	13.5	0.15	24.0	0.70	40.2	3.3
59	27.0	90.1	26.9	34.2	7.8	20.9	16.0	0.15	28.0	0.80	43.4	3.3

Remarks

THD-80

Table No. 20

Lab. No. 47-136-E County Bastrop Hwy. 290 Project

Material Sand - Clay Identification

Description Typical Sand - Clay hauled from cuts for Select Material

Opt. Moist. 12.9 Opt. Dry Density 114.4 at Comp. Effort 13.26 ft.lb./cu.in.

Spec. No.	Molding Data		Curing Data Moist. Cont.				Stress-Strain Relations					
							Straight Portion				Ultimate	
							From		To			
	% Moist. Dry Weight	Dry Den. Lbs./Cu.Ft.	% After Dry.	% Abs. After Capillarity	% Vol. Swell	Appl. Lat. Pres. P.S.I.	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain	PSI Unit Stress	% Strain
23	12.8	114.5	6.3	13.7	Small	3.7	11.5	0.75	14.5	1.10	41.3	5.05
25	12.9	114.7	6.0	13.8	"	5.7	14.0	0.70	20.0	1.10	54.3	5.85
24	12.9	114.1	6.4	13.9	"	8.2	18.0	0.70	23.0	0.90	64.0	5.82
26	13.0	114.1	6.6	13.9	"	10.7	20.0	0.60	30.0	1.00	77.9	6.60
27	12.9	114.8	5.9	13.7	"	15.7	24.0	0.50	38.0	0.85	103.6	7.28
28	12.9	114.4	6.4	13.9	"	20.7	14.0	0.0	50.0	0.80	125.8	6.22

Remarks

APPENDIX

TRIAXIAL COMPRESSION TEST, TEXAS HIGHWAY DEPARTMENT, THD-80

FOREWORD

This triaxial compression test procedure has been developed on the basis of studies pertaining to the factors which influence the strength test results. The test method is applied, Part I, to remolded specimens of disturbed materials such as soils or aggregate bearing materials with a top size of 2 inches for the largest particles, and Part II, to undisturbed cores from roadway or foundation material from the site of a proposed structure.

Figure 1.

APPARATUS

1. Apparatus used in T. H. D. Testing Procedures No. 53 and 83.
2. Axial Cells — light weight stainless steel cylinders; inside diameter $6\frac{3}{4}$"; fitted with a standard air valve; inside of each cylinder is a tubular rubber membrane, 6 in. diameter. See Figure 1 (small cells for bridge cores).
3. Aspirator or other vacuum pump.
4. Constant pressure air supply. See Figure 2.
5. Air compressor or suitable pump.
6. Auxiliary compressed air storage tank with pressure gauges, valves, and air lines. See Figure 4.
7. Light weights for surcharge loads.
8. Micrometer dial, calibrated in 0.001 in., with support.
9. Dial housing to transmit load and cylindrical loading block. See Figure 4.
10. Calibrated proving rings.

PART I

TEST PROCEDURE FOR DISTURBED REMOLDED MATERIALS

Preparation of Sample.

Prepare the material according to the method given in THD-53, Part II.

Moisture-Density.

Use the method outlined under THD-83 and determine the optimum moisture and the maximum density. The compactive effort specified for the type of material being tested should be used. Store specimens in damp room if it is desirable to use them in test.

Specimens Molded at Optimum Moisture for Testing

1. Mold six specimens 6" in diameter and 8" in height at the optimum moisture and density. These specimens should be made as nearly the same as possible. Any test series of specimens, along with the ones made for the moisture-density curve, should be molded at the same compactive effort. If it is desirable to try different compactive effort, a complete new set of specimens must be molded.
2. Immediately after extruding from the mold, the specimens with top and

bottom stones in place are stored in the moist room over night. The purpose of this over-night storage is to permit the moisture to equalize in all parts of the specimens; therefore, the specimens should not come in direct contact with capillary or surface water. Record data on form shown in Figure 5.

Figure 2.

Drying of Specimens

After the molding moisture has had time to equalize, the specimens are returned to the laboratory and dried according to the type of material.

1. In the case of a flexible base material that will not develop shrinkage cracks, the specimens are placed in a drying oven for 8 hours (about one-half of the molding moisture will be removed) at a temperature of 140 F. These specimens should be cooled for at least 8 hours before following the next step (capillarity).

2. For a doubtful material, it might be placed in a drying oven (140 F.) and checked frequently for the appearance of shrinkage cracks. If cracks appear, some drying at room temperature may be allowed during the cooling period.

3. In the case of very plastic clay subgrade soil that will crack badly while shrinking, air dry at room temperature until cracks develop.

Excessive cracking (in steps 2 and 3) will damage the test specimens.

Subjecting Specimens to Capillarity

1. The specimens are ready to be weighed, measured and prepared for capillarity. The porous stones are not to be removed from the specimens for any reason until after the specimens have been tested.

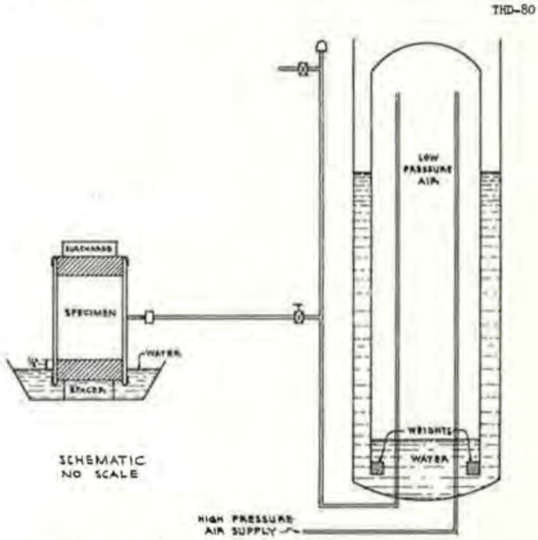


Figure 3. Diagram of equipment used during capillary wetting.

2. Each specimen, with porous stones in place, is wrapped with a piece of slitted filter paper and enclosed in a pressure cell. This is done by applying a partial vacuum to the pressure cell, slide it over the specimen and release vacuum.

3. Place the assembly in a pan and adjust the water level on the lower porous stone to a distance of one-half inch below the bottom of the specimens. See Figure 3 for schematic drawing of set up.

4. The cell is then connected to the constant pressure air manifold and subjected to the usual lateral pressure of 1 psi. This constant pressure shall be maintained throughout the period of capillarity.

5. Next, place an appropriate vertical surcharge weight (which will depend upon the condition of the proposed use of the material) to the upper porous stone. For

flexible base materials use one-half pound per square inch and for subgrade soils use one pound per square inch. The upper porous stone is considered as part of the surcharge weight.

6. After the specimens have been in capillarity for (10 days for materials having a P. I. below 15) a period of time in days equal to the P. I. for all materials with P. I. above 15, they are prepared for testing. The specimens should be kept in a damp room during the time they are subjected to capillarity. See Figure 2. Record data as shown in Figure 6.

Preparing Specimens for Testing.

1. Disconnect air hose from cell and remove surcharge weight. Return specimens to laboratory for testing.

2. Remove cell from specimen and discard filter paper.

3. Measure the circumference of the specimens by use of a tape or strip of paper. Measure the height of specimen including the stones, and record as "height out of capillarity". Record the height of each stone.

4. Weigh specimens in order to obtain the percentage of absorption.

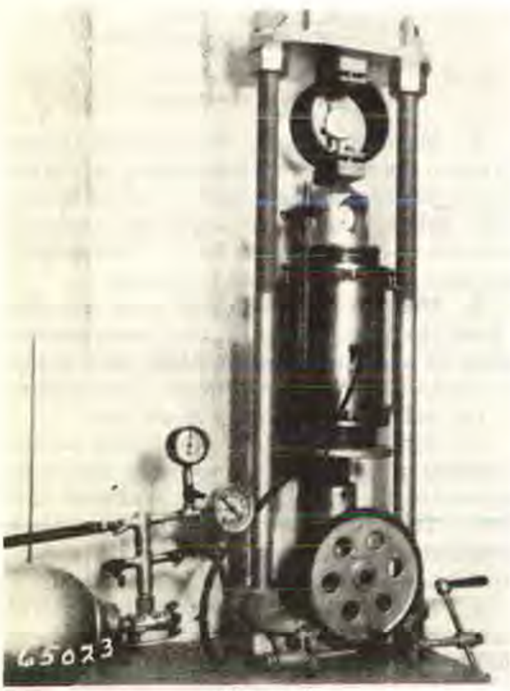


Figure 4.

TRIAXIAL COMPRESSION TEST DATA SHEET
FOR COMPACTING SPECIMENS

TID-80

Sample #						
Date Molded						
Compactive Effort						
Total % H_2O						
Pounds Air Dry						
Pounds H_2O Desired						
Less Pounds Hygro.						
Wt. Water & Jar						
Tare Wt. of Jar						
Mold No.						
Wet Wt. Spec. & Mold						
Tare Wt. Mold						
Wet Wt. Spec.						
Height Mold						
Dial Reference						
Dial Reading						
Height Specimen						
Vol. per Lin. In.						
Vol. of Spec.						
Wet Density Spec.						
Dry Wt. Pan & Spec.						
Tare Wt. Pan						
Dry Wt. Mat'l.						
Wt. H_2O						
% H_2O on Total						
Dry Density						
Remarks:						

Figure 5.

5. Replace the axial cells on the specimens and they are ready to be tested.

Testing the Specimens

Record results as shown in Figure 7. In brief, the specimens are tested in compression while being subjected to their assigned constant lateral pressure. The rate of deformation to use is 0.15 inch per minute. Simultaneous readings of load and deformation are to be taken at intervals of 0.01 inch deformation. The procedure described below applies to both a hand operated or motor driven gear press equipped with a deformation gauge and a proving ring. See Figure 4.

1. Disengage the worm gear drive and crank the press down far enough to have room to place specimen and loading blocks in the press.

2. Center specimen (with upper and lower loading blocks in place) in press. Adjust the deformation gauge in such a manner that it will be down against the center of top loading blocks and also compressed for almost the total length of travel of the stem. The gauge must be placed in this position because during the compression of the specimens it moves away from the gauge. Set dial on the de-

TRIAXIAL COMPRESSION TEST DATA SHEET
FOR CAPILLARITY AND VOLUME CHANGE
THD-60

Sample #						
Cell #						
Lbs. of Added surcharge						
Date Molded						
Date in air dryer						
Date in capillarity						
Date out capillarity						
Height in capillarity						
Height out capillarity						
Difference in height						
% swell						
Weight after oven air dry						
Dry weight stones & plate						
Dry cell						
Dry filter paper						
Oured dry weight sample						
Dry weight sample						
Weight moisture in sample						
% moisture to capillarity						
Weight after capillarity						
Weight wet stones & plate						
Weight wet cell						
Weight wet filter paper						
Weight wet sample						
Dry weight sample		*				
Weight moisture in sample						
% moisture after capillarity						
Remarks:						

Figure 6.

formation gauge to read zero or on an even tenth.

3. Set the special bell housing over the deformation gauge but do not allow it to touch the gauge or its mounting. Raise the press by means of the hand crank, center and set the ball on the housing into the socket of the proving ring. It should be noted that the compressive stress will necessarily be applied along a vertical line through the center of the ball mounted in the bell housing. Read the deformation gauge and record as deformation under dead load.

4. Connect the air line to the axial cell and apply lateral pressure to the specimen. The usual lateral pressures used for a series of tests are 0, 3, 5, 10, 15 and 20 psi. The lateral pressure applied by the air will tend to change the initial reading of the deformation gauge. Load the specimen as the air pressure is applied until the deformation gauge reads the same as recorded in step 3, above. The proving ring gauge is now read and the reading entered in the load column opposite the initial deformation reading.

5. The actual test is now ready to be run. The dial gauge on the proving ring is read for each 0.01 of an inch deforma-

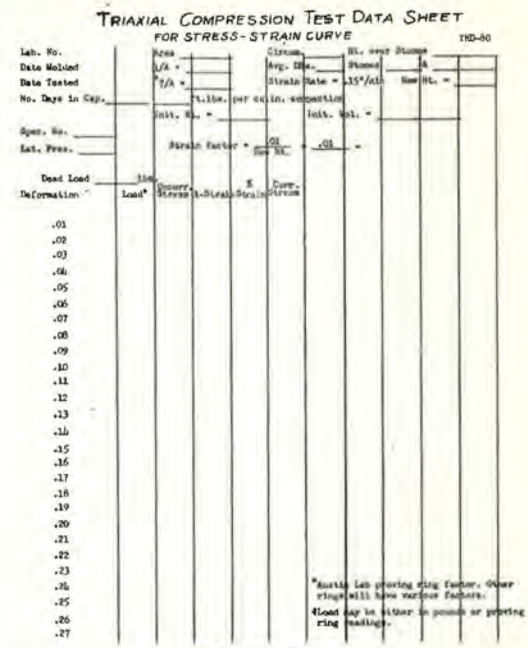


Figure 7.

tion. This procedure is continued to the point of failure which is reached when the needle on the proving ring gauge remains the same or drops off.

6. All of the above procedure applies to the unconfined specimen except that no air or axial cell is used.

7. The specimens are broken up, then placed in tared pans and dried to constant weight at 230 F. The pans of

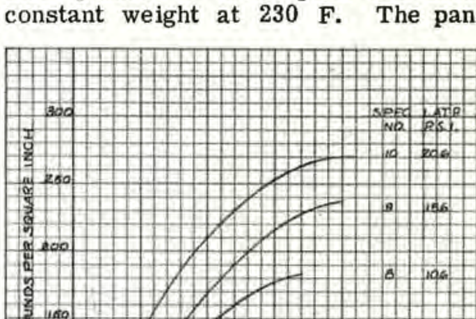


Figure 8. Stress-strain diagram.

material are reweighed to obtain the dry weight. The stones are weighed, dried, and the dry weight obtained.

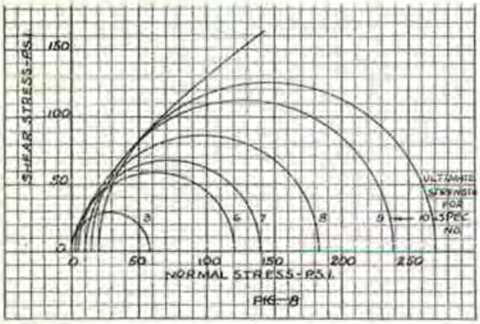


Figure 9. Mohr's diagram.

Calculations

1. Calculate molding moisture and dry density of each specimen.

2. Compute the percentage of swell and absorption after capillarity.

$$\% \text{ Volumetric Swell} = \frac{(\text{Volume Change})}{(\text{Molded Volume})} 100$$

3. Calculate data for stress-strain curve.

For each individual specimen tested, terms and symbols are defined as follows:

- P = total vertical load on the specimen at any given deformation, expressed in pounds. It is the sum of the applied load measured by the weighing mechanism plus the dead weight of the upper stone, loading block and dial housing.
- A = the original area of cross-section of the specimen expressed in square inches calculated from measured diameter or circumference = $\pi (\text{radius})^2$.
- h = the original height of the specimen in inches, measured before beginning the test.
- d = the total vertical deformation at the given instant, measured in inches by the deformation dial.
- %S = percent strain - the relation of the deformation at the given instant to the original height, expressed in percent, calculated from the formula $\% S = 100 \times d/h$.
- P/A = the nominal vertical unit stress at any given deformation expressed in pounds per square inch.
- p = the corrected vertical unit stress in pounds per square inch. A correction is necessary because the area of the cross-section increases as the specimen is pressed down. Assuming that the specimen deforms at constant volume, $p = P/A \times (1 - \% S/100)$.

Plotting Curves and Diagrams

1. Plot molding moisture against dry density as shown in THD-83.

2. Plot stress-strain curve as shown in Figure 8 for each increment of deformation. This is not mandatory but desirable.

3. The Mohr's diagram of stress (Figure 9) is constructed upon coordinate axes in which ordinates represent shear-stress and abscissae represent normal stress, both expressed as pounds per square inch and both drawn to the same scale.

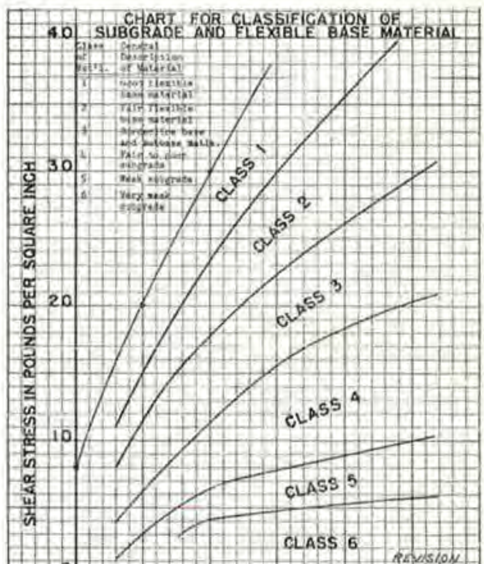


Figure 10.

σ_{III} = the minor principal stress which is the constant lateral pressure applied to the specimen during an individual test, and σ_I = the major principal stress which is the ultimate compressive strength (the ultimate value of p) of the specimen at the given lateral pressure. Each individual test will be represented by one stress circle constructed as follows:

On the axis of normal stress plot σ_{III}.

On the same axis plot σ_{I}.

Construct a semi-circle on the abscissa base line with its center at a distance equal to $(\sigma_{I} + \sigma_{III})/2$ from the origin, with its radius equal to $(\sigma_{I} - \sigma_{III})/2$ and cutting the base line at σ_{I} and σ_{III}.

Classification of Material and Interpretation

Transfer the rupture envelope on to the chart for classification of subgrade and flexible base material (Fig. 10) and classify.

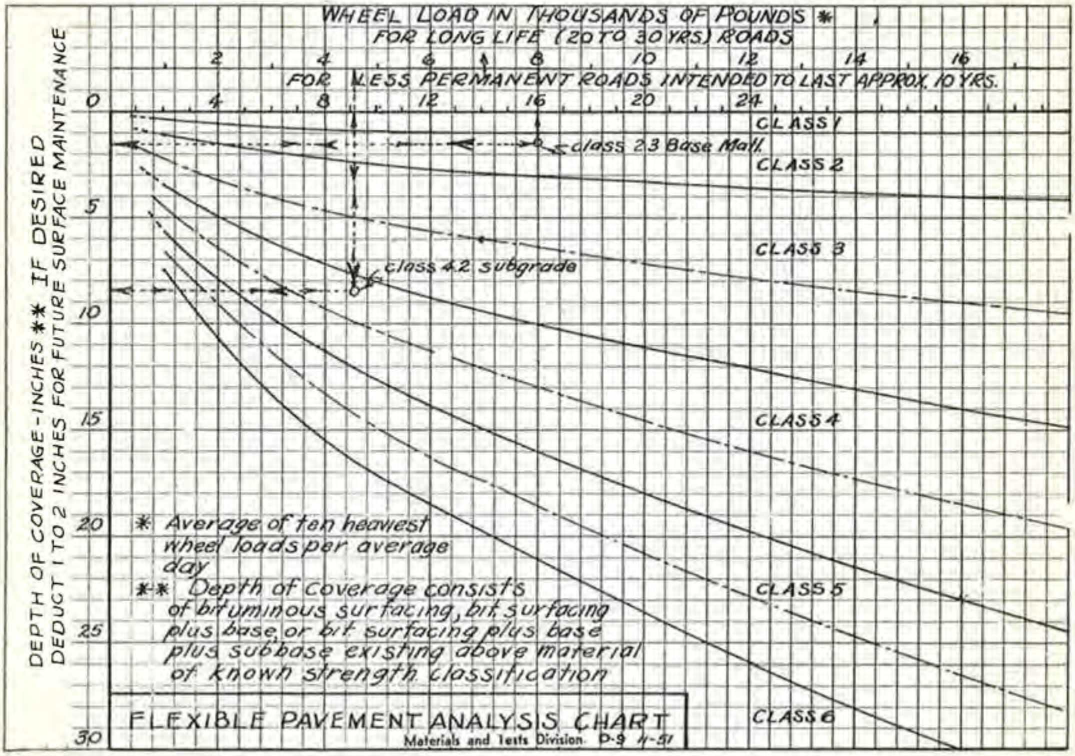


Figure 11.

Repeat these steps for each specimen tested. The test procedure calls for a minimum of 5 specimens, each tested at a different lateral pressure, in order to provide data for enough stress circles to define the rupture envelope on the Mohr diagram.

Draw a line tangent to all of the stress circles. This line, called the "Rupture Envelope", represents the shearing strength of the material under various conditions. It is practically impossible to avoid compacting an occasional specimen that is not identical with the other specimens in the same set with regard to moisture, density and particle arrangement. Triaxial test results from such specimens may be out of line. In drawing the rupture envelope, disregard any stress circles that obviously are out of line with others of the same set.

There appears to be no necessity for further differentiation of Class 1 materials. However, in Classes 2 through 6 the rupture envelope may fall between class limits so that interpolation is necessary to determine the depth of cover required. Between class boundaries the weakest or most critical point on the rupture envelope governs, i. e., the point nearest the lower boundary of the class.

Using the classification of material and the design wheel load, determine the depth of flexible base from flexible pavement analysis chart (Fig. No. 11), Careful consideration of road service life should be made and only long life design should be used in urban sections.

Reporting of Test Results

As a minimum, a report should include

the identification, soil constants and gradation on Form 476A, a summarization of data in the attached tabulation (Fig. 12) and a classification plotting on Figure 10.

Note: For control of compaction see THD-110.

THD-80

Table No. __

Lab. No. ________ County ________ Hwy. ______ Project ________

Material ________ Identification ________

Description ________

Opt. Moist. _____ Opt. Dry Density _____ at Comp. Effort _____ ft.lb./cu.in.

Spec. No.	Molding Data		Curing Data Moist. Cont.		% Vol. Swell	Appl. Lat. Pres. P.S.I.	Stress-Strain Relations					
							Straight Portion				Ultimate	
							From		To			
	% Moist. Dry Weight	Dry Den. Lbs./Cu.Ft.	% After Dry.	% Abs. After Capillarity			Stress	% Strain	Stress	% Strain	Stress	% Strain

Remarks

Figure 12.

PART II

TEST PROCEDURE FOR UNDISTURBED CORES FROM ROADWAY OR BRIDGE FOUNDATION

Preparation of Specimens for Testing

1. Remove metal or cloth and paraffin protective cover and examine core. Classify core as to type of material, color, structure, and moisture content.

2. Cut off a piece of the core, the length of which shall be at least twice the diameter of finished test specimen. Coat this specimen and remainder of core with paraffin to prevent loss of moisture.

3. By means of a knife, saw or other cutting tools, make the ends parallel to each other and square with the center line of the specimen.

4. Place a circular porous stone on the top and bottom of specimen. In cases where the specimen is larger than the stones or if the sides are irregular, trim sides of specimen flush with the top and bottom stones.

5. Immediately after trimming the cylindrical specimen, weigh, measure circumference and height, and place specimen in testing cell. If the lateral pressure to be applied during testing is above 25 pounds per square inch, fasten retainer rings on the pressure cell.

6. Place the specimen in compression machine and apply lateral pressure (one pound p. s. i.) for each foot of overburden depth and additional increments of 5 to 10 p. s. i. over the range of the lateral pressures expected to develop due to loading.

7. Test the specimen by increasing the deformation load to failure. This load should be applied at a rate of 0.15 inch deformation of specimen and readings taken at each 0.01 of an inch deformation.

8. Release the load and remove specimen from pressure cell. Examine the break in the specimen.

9. Dry specimen at 230 F. and obtain dry weight.

10. Calculate dry density of core in pounds per cubic foot.

Interpretation

Prior to testing, undisturbed cores should be in equilibrium with respect to moisture and density under the conditions of proposed use. If cores are taken at a relatively great depth to represent the subgrade of flexible base in a proposed cut (not to be cut below grade and recompacted), the cores should be saturated under the proposed new surcharge before testing.

Relatively shallow cores, representing the subgrade under an existing, unbroken base, probably are in equilibrium and require no processing. If these shallow cores are from an uncovered subgrade, they should be subjected to capillary wetting under the proposed surcharge. A rather large number of cores may be required under the latter condition to arrive at average values. When cores representing subgrades of flexible pavements have been processed and tested, the rupture envelopes may be superimposed on the classification chart shown and may be judged according to the wheel loads and base depths shown in Figure 11.

Application of Strength Versus Stress for Foundations

Figure 13 shows the plotting of Mohr's diagram of stress representing the strength tests on the undisturbed cores from the area of the footing. Superimposed on the diagram is a stress circle (Circle A) representing the total vertical and radial stresses at the critical point on the axis of the loaded area. These stresses were estimated as shown in the upper part of Figure 13 and in the "Example of Application" in the latter part of this report. The stress line, shown in Figure 13 as a dashed line, is tangent to this circle and is constructed in accordance with step 2, below. All stress circles representing various loadings on this size footing and subject to this particular surcharge will be tangent to the stress line. The factor of safety against failure of the soil is assumed to be the ratio; ultimate load soil will support divided by the applied load. This factor can be calculated from soil mechanics formulae if definite values of cohesion and angle of internal friction are known. In order to determine definite values of cohesion and friction, it is necessary that Mohr's envelope of failure be a straight line. Since this is not the case for many soils foundation materials, the use of a graphical method is preferred. The procedure for determining this factor of safety is as follows:

1. Draw the rupture envelope representing the strength of the soil foundation material.
2. Beginning at a point on the normal stress axis, representing the hydrostatic surcharge (U_Z from Fig. 13), draw a straight line making an angle of 33. 8 deg. with the normal stress axis.
3. By trial and error, find the stress circle, if any, that is tangent to both the 33. 8-deg. line and the rupture envelope.
4. Read the major principal stress (or read minor principal stress).
5. Subtract the hydrostatic surcharge U_Z from the major principal stress and divide by 0. 808 (or subtract U_Z from minor principal stress and divide by 0. 230).
6. The ratio of the load determined in Step 5 to the applied load is the factor of safety with respect to load.

In designs made with factors of safety of $1\frac{1}{2}$ to $2\frac{1}{2}$ there have been no excessive settlements in the structures so designed. The above factor of safety becomes infinitely great when soil foundation materials have angles of internal friction in excess of approximately 34 deg.

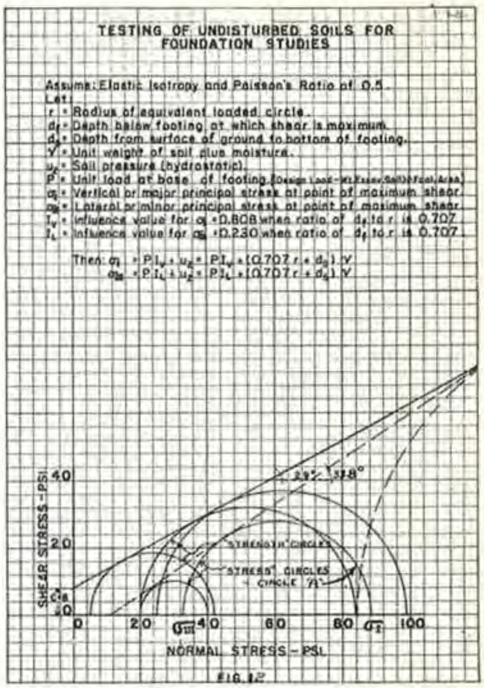


Figure 13.

Example of Application to Foundation Design

The following example is based on data obtained for a grade separation structure where the highway passes underneath the railroad. It is desired to place footings approximately ten feet below the highway grade line in a sand-clay cut which has a wet unit weight of 125 lb. per cu. ft. This probably is the minimum depth of penetration because the roadway ditches will drain both surface and underground water. The problem is to determine the size of an adequate spread footing for this structure which has a pier load of 660 tons. Since only a few footings are needed, underreamed footings probably are not feasible. Assuming that each pier is to rest on two, 7-foot square footings on 21-foot centers, and the test results showing Mohr's envelope of failure for the soil involved are as shown in Figure 13, the problem may be solved as follows:

1. The radius "r" of a circle equivalent

Texas Highway Department
Form 476A 7-51-5m

84005-758-5m

SOILS AND BASE MATERIALS TEST REPORT

Laboratory No. ________
Date Rec'd ________ Reported ________
Engineer ________
Address ________
Contractor ________
Sampler H. S. Gillette & C. McDowell
Sampler's Title ________
Sampled From Roads
Producer ________
Quantity Represented by Sample ________
Has been Used on ________

Control Number ________ Section Number ________ Job Number ________

County ________ Federal Project No. ________ Highway No. ________

16
District No. ________ I.P.E. No. ________ Req. No. ________ Date Sampled ________

Specification Item No. ________
Material from Property of ________

Proposed for Use as ________

Lab. No.	LL	PI	FME	CME	SL	LS	SR	Class	Soil Binder	% Moist.	Str. Class
50-76-R	32	18	23	-	19	6.8	1.74	A-6-2	26	- --	1.0
77	40	23	25	-	16	11.5	1.86	A-6	100	14.8	4.9
72	38	15	30	-	25	6.0	1.61	A-4-7	49	--	3.1
73	30	15	27	-	17	6.6	1.80	A-2	89	9.4	3.4
74	49	22	34	-	28	8.9	1.52	A-7	45	--	3.3
75	56	35	28	-	14	17.6	1.90	A-7	98	18.1	4.8

PERCENT RETAINED ON

Lab. No.	Round Opening Screens — Opening in Inches									Square Mesh Screens — Sieve Numbers						Grain Diam. in Millimeters			Specific Gravity
	3½	3	2½	2	1½	1	¾	½	¼	10	20	40	60	100	200	.05	.005	.001	
50-76-R					4	11	16	25	44	59	71	74	75	77	88	89	93	97	2.64
77													1	3	32	45	73	87	2.65
72					2	4	8	12	22	33	43	51	56	61	67	70	90	98	2.64
73									1	2	3	11	25	40	65	73	90	96	2.60
74					1	4	9	15	27	42	51	55	59	62	67	70	93	97	2.65
75											0	2	9	25	51	56	66	83	2.66

SAMPLE IDENTIFICATION

Lab. No.	Identification Marks	Location—Properties—Station Numbers	Type of Materials
50-76-R	No, 1, FM-136	Refugio Co. (between Bayside and Woodsboro)	5½" of shell base
77	" "		Subgrade soil
72	No. 3 US 59	Business route in Mathis, 3 blks.N. int. of business rt. and regular US 59	10" of caliche base
73	" "		Subgrade soil
74	No. 4, St. 202	3 Mi.E. of Beeville	7" of caliche base
75	" "		Subgrade soil

Exhibit A.

Texas Highway Department
Form 476A 7-52-5m

84005-752-5m

SOILS AND BASE MATERIALS TEST REPORT

Laboratory No. ______
Date Rec'd ______ Reported ______
Engineer ______
Address ______
Contractor ______
Sampler H. S. Gillette & C. McDowell
Sampler's Title ______
Sampled From Roads
Producer ______
Quantity Represented by Sample ______
Has been Used on ______

Control Number ______ Section Number ______ Job Number ______
County ______ Federal Project No. ______ Highway No. ______
District No. ______ I.P.E. No. ______ Req. No. ______ Date Sampled ______
Specification Item No. ______
Material from Property of ______
Proposed for Use as ______

Lab. No.	LL	PI	FME	CME	SL	LS	SR	Class	Soil Binder	% Moist.	Str. Class
50-82-E	34	20	22	-	16	9.3	1.91	A-6-2	27	-	3.1
83	24	8	18	-	17	4.0	1.79	A-2	96	8.2	3.1
78	31	14	22	-	18	6.8	1.82	A-2	36	-	2.8
84	82	56	49	-	12	25.8	2.05	A-7	86	23.6	5.2
80	29	15	21	-	16	6.8	1.87	A-2	31	--	2.7
81	66	41	39	-	15	20.2	1.90	A-7	96	24.6	4.7
49-40-R	24	7	20	-	17	4.0	1.78	A-2	32	--	3.0
39-11-MR	74	45	45	-	12	24.0	2.00	A-7	97	31.	4.9

PERCENT RETAINED ON

Lab. No.	Round Opening Screens — Opening in Inches									Square Mesh Screens — Sieve Numbers						Grain Diam. in Millimeters			Specific Gravity	
	3½	3	2½	2	1½	1	¾	½	¼	10	20	40	60	100	200	.05	.005	.001		
50-82-R				3	7	19	27	40	56	67	72	73	76	79	83	85	92	97	2.70	
83									1	2	3	4	7	27	63	69	83	92	2.60	
78					8	19	28	37	51	59	62	64	66	67	69	71	90	98	2.65	
84							0	5	8	12	13	14	14	16	22	27	61	83	2.66	
80					3	14	22	34	47	59	64	69	75	80	84	86	98	99	2.75	
81										1	3	4	5	7	16	20	63	83	2.71	
49-40-R					8	17	21	28	38	51	62	68	73	82	90				2.60	
39-11-MR												3	3	4	5	12	54	77	2.71	

SAMPLE IDENTIFICATION

Lab. No.	Identification Marks	Location—Properties—Station Numbers	Type of Materials
50-82-R	No. 5 St. 346	Bexar Co. Sta. 888,12 Mi.S.Int.Loop 13 and 346	13" Gravel Base
83	" "		Subgrade Soil
50-78-R	No. 6 Loop 13	2000' N.Main gate,Lakcland Air Base	13" Gravel Base
84	" "		Subgrade Soil
50-80-R	Np. 7, Loop 13	2000' E. of Kelly Field Overpass	12" Gravel Base
81	" "		Subgrade Soil
49-40-R	US 290 Travis Co.	East of Manor	10" Gravel Base
39-11-MR	" "		Subgrade Soil.

Exhibit B.

Texas Highway Department
Form 476A 7-58-5m

84005-753-5m

SOILS AND BASE MATERIALS TEST REPORT

Laboratory No.
Date Rec'd Reported
Engineer
Address
Contractor
Sampler H. S. Gillette & C. McDowell
Sampler's Title
Sampled From
Producer
Quantity Represented by Sample
Has been Used on

Control Number / Section Number / Job Number
County / Federal Project No. / Highway No.
District No. / I.P.E. No. / Req. No. / Date Sampled
Specification Item No.
Material from Property of
Proposed for Use as

Lab. No.	LL	PI	FME	CME	SL	LS	SR	Class	Soil Binder	% Moist.	Str. Class
50-90-R	35	18	23	-	18	8.5	1.76	-	48	-	3.1
91	34	19	22	-	16	8.7	1.86	A-7-4	100	9.2	4.6
92	35	17	26	-	20	7.6	1.75		59	-	3.4
93	36	20	25	-	15	10.5	1.87		100	10.6	4.4
49-14-R	22	6	19	-	15	4.1	1.89		28	-	1
47-137-E	30	7	23	-	25	2.7	1.59	A-2	98	-	3.4

PERCENT RETAINED ON

Lab. No.	Round Opening Screens — Opening in Inches									Square Mesh Screens — Sieve Numbers						Grain Diam. in Millimeters			Specific Gravity
	3½	3	2½	2	1½	1	¾	½	¼	10	20	40	60	100	200	.05	.005	.001	
50-90-R				0	7	19	24	30	37	44	49	52	54	63	77	80	90	98	2.64
91													3	10	22	39	78	92	2.68
92					2	5	8	13	24	33	38	41	43	50	63	70	88	97	2.65
93													2	11	27	48	78	92	2.65
49-14-R					13	25	31	39	48	56	65	72							
47-136-E											0	2	5	53	74	81	84	87	2.63

SAMPLE IDENTIFICATION

Lab. No.	Identification Marks	Location—Properties—Station Numbers	Type of Materials
50-90-R	No. 14 US 70	Floyd Co., 4 Mi.E. of Hale Co.Line	9" Caliche Base
91	" "	Sta. 330+15	Subgrade Soil
50-92-R	No. 15, US 62	Crosby Co. 1 Mi. S. Floyd-Crosby	7" Caliche Base
93	" "	Co. Line	Subgrade Soil
49-14-R	US 290	8" Crushed Stone Base	Flexible Base
47-136-E	"	Sampled from R.O.W. Cut	Subgrade Soil and Select Material

Exhibit C.

in area to a 7-foot square is equal to 3.95 feet.

2. From Figure 13:
   $d_f = .707 \times 3.95 = 2.8'$

   $d_s = 10'$

   P = 330 tons plus weight of concrete pier minus weight of excavated soil,
   = 330 x 2000 + (7 x 7 x 10) (150-125) divided by 49
   = 13719 lb. per sq. ft. = 95.3 psi.

   $I_V = .808$ (See Fig. 13)

   $I_L = .230$ (See Fig. 13)

σ_I = 95.3 x .808 + 12.8 x 125 divided by 144 = 88.1 psi.

σ_{III} = 95.3 x .230 + 12.8 x 125 divided by 144 = 33.0 psi.

Factor of safety = $\frac{84-11.1}{.230}$ divided by applied load P (95.3 in this case) = 3.3

Approximate factor of safety for 6' x 6' footing = $(\frac{36}{49})$ 3.3 = 2.4

Approximate factor of safety for 5' x 5' footing = $(\frac{25}{49})$ 3.3 = 1.6

Therefore, size of footing should be 6' x 6', or greater.

DISCUSSION

RAYMOND C. HERNER, Chief, Airport Division, Technical Development and Evaluation Center, Civil Aeronautic Administration — Although we probably never shall eliminate the factors of experience and engineering judgment in consideration of any design problem, it is refreshing to note the increased use of strength tests and definite design criteria by the various highway departments. The Texas department has been one of the leaders in this movement. For several years it has had a flexible pavement design method based partly on theoretical concepts and partly on experience, with the triaxial test used for evaluation of subgrade and base course materials.

McDowell's excellent report now gives us an opportunity to see how well this design method is working. The results thus far are encouraging.

The writer has one suggestion in regard to evaluating traffic which might help in coordinating laboratory findings with service experience. There are many pavements which have stood up for years under comparatively light traffic, then have failed within a comparatively short period of time as loads increased. If the highest wheel loads are then averaged for the entire life of the pavement, we are likely to end up with a deceptively low average value for the load which caused the destruction.

It is the old problem of adding apples to oranges. Despite continuing efforts to do so, it is impossible to add heavy loads to light loads with any result except confusion. It would appear desirable then to average only loads within a comparatively narrow weight range in order to determine the critical load for design or evaluation purposes. Although frequency of loading must be considered also, it should not be allowed to obscure the effect of heavy loads.

McDowell's paper was particularly valuable because of the detailed test data given in the appendix. The writer found it interesting to compare the strength ratings of the materials as determined by the Texas test method with their ratings as determined by the CAA soil-classification method. The latter method is based entirely on mechanical analysis, liquid limit, and plasticity index. By plotting values from the two systems against each other, it was possible to establish a rough correlation as shown in Table A.

TABLE A

CAA Classification	Texas Classification
E-1	1
E-2	1.6
E-3	2.1
E-4	2.7
E-5	3.25
E-6	3.75
E-7	4.25
E-8	4.6
E-9	4.8
E-10	5.0
E-11	5.1
E-12	5.2

The correlation is fairly good for subgrade soils but is not so good for base course materials. This is not surprising, as the CAA classification is intended for evaluation of materials for subgrades rather than base courses.

It also was interesting to compare the Texas materials with those tested triaxially in the CAA's Load-Transmission Project (now being operated through the cooperation of the Navy Bureau of Yards and Docks). The strength range and characteristics of materials used in the Texas roads are comparable to those of materials used in the CAA tests if due allowance is made for the difference in height-diameter ratio of the triaxial specimens. The Texas Department of Highways uses a 6-by-8-inch specimen ($H/D = 1.33$) while the CAA uses a 10-by-20-inch specimen ($H/D = 2.0$) as standard.

A limited number of tests have been run by the CAA in order to determine the effect of specimen height. Average results for gravel and sand are given in Table B.

TABLE B

Height/Diameter Ratio	Indicated Strength Percentage of Standard Specimen
0. 8	210
1. 33 (Texas)	123
1. 6	111
2. 0 (CAA Std.)	100
3. 0	86

It seems quite likely that the effect would be less pronounced on tests of cohesive soils, but would be greater on very coarse granular materials.

The writer is very pleased to have access to the information given in McDowell's paper, and hopes that further papers will be forthcoming from the same source as additional service experience is accumulated.

Flexible-Pavement Design as Revised for Heavy Traffic

L. D. HICKS, Chief Soils Engineer,
North Carolina State Highway and Public Works Commission

● AT the Twenty-Sixth Annual Meeting of the Highway Research Board December (1946), the writer presented a paper "Current Base Design Practices in North Carolina" in which a procedure for the rational design of a flexible pavement was given. This paper presents a revision of the original method made necessary by the fact that the thicknesses derived by use of the original method were found to be inadequate for axle loads in excess of 13,000 lb. The pavement thicknesses determined by the original design method have been adequate for axle loads below 13,000 lb., for reasons that will be given later, so the thicknesses determined by this method are still used for these lighter loads. The revised method is used for the determination of the pavement thicknesses for the heavier loads. Both methods will be explained in detail in this paper and certain recommendations made relative to base and surface types used for heavy loads.

ORIGINAL METHOD

The rational design of a flexible pavement involves three important factors that must be given consideration: (1) the magnitude and nature of the applied load and the mode of its application: (2) the strength or bearing capacity of the subgrade soil as well as the bearing capacity of the components of the pavement structure (subbase, base, and surface course); and (3) the distribution of the pressure exerted at the surface of the pavement by the applied load.

The selection of the design load to use in the flexible-pavement-design problem requires considerable judgment on the part of the design engineer. A few loads heavier than the load for which a flexible pavement was designed will not cause its failure. Failures in flexible pavements do not occur instantly but gradually, and many fairly constant load applications are necessary to produce objectionable distortion or complete failure. Consideration must be given to the possibility of the traffic increasing in volume as well as in weight at some time in the near future. Also, loads heavier than the design load will not be detrimental if the heavier loads use the pavement only during periods when the subgrade moisture is low (1). A factor of safety is not usually considered necessary in flexible-pavement design, but it is advisable and may be considered good practice to overdesign slightly by selecting a design load that is well within, or slightly above, the axle loads of appreciable occurrence that are expected to use the road.

It is the practice in North Carolina to use four design loads, 8,000 lb. for the lightest design, 13,000 lb. for the medium, 18,000 lb. (legal limit with a 1,000-lb. tolerance) for the heavy, and 20,000 lb. for the heaviest. A pavement is rarely designed for loads intermediate between these.

In the original METHOD ~~article~~ the pressures exerted by the various wheel loads were determined from the inflation pressures obtained when truck-weight surveys were conducted. These pressures were increased 10 percent for tire-wall stiffness. The areas of contact were obtained by dividing the wheel loads by the inflation pressures from which diameters of circles of equal areas were calculated. The revised method of design, ~~described later in this paper,~~ uses a different procedure for determining the pressures and diameter of contact area. GO TO PAGE

The strength of materials is measurable by test, but the values obtained are not always the proper ones to use in the design of a pavement. Many of the test methods used do not give values that may be substituted in formulas derived by sound mechanics, and many of the methods designed to give such values produce values that are incorrect, because the test is performed on the material in a condition different from that under which it must serve.

Soils consist primarily of mineral particles which may vary in size from large stone or gravel to particles of microscopic

TWO METHODS FOR DETERMINING NECESSARY PAVEMENT THICKNESS ARE DESCRIBED.

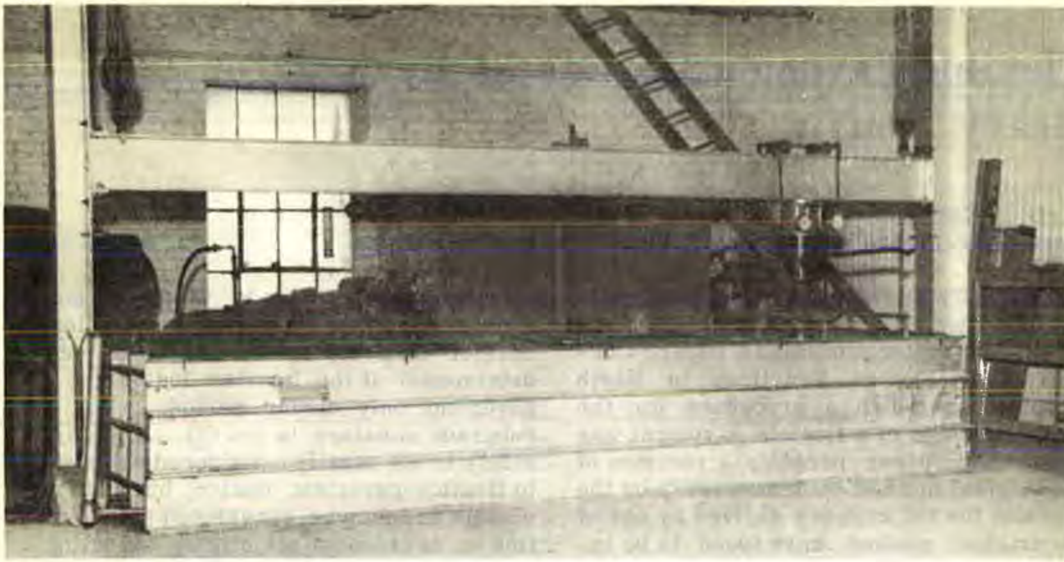


Figure 1.

size. When separated, the constituents are called coarse aggregate, sand, silt, and clay, depending upon their size, and each constituent possesses certain inherent characteristics when in the presence of water. The characteristics of mixtures of these constituents vary also in the presence of water, depending upon the relative amount of each. Adsorbed ions on the clay fraction, as well as the type of the clay mineral itself, influences the character of the clay in the presence of water and, consequently, the characteristics of the soil mixture of which it is a part.

Figure 2.

The water content of a given soil is the most-effective agent influencing its strength or bearing capacity, and in service the moisture content will fluctuate, although the soil be protected by an impervious surface course (1). The proper moisture content at which to test the soil for bearing capacity is the highest average moisture content that the particular soil will have in service. This moisture content varies with the soil, climate, type of surface, and maybe other factors, and should be determined by careful investigation. Such an investigation has been carried out in North Carolina and reported at the annual meeting of the Highway Research Board in 1948 (1). In this report the moisture contents are expressed in terms of the optimum moisture for compaction of the soil at which various soils should be tested for bearing capacity. These moisture contents are for soils as they exist in service in North Carolina and are not necessarily correct for other localities.

Since the bearing capacity of soils change with fluctuations in moisture content, load tests made on existing subgrades do not give values that should be used in design, unless the subgrades happen to be in the proper condition. This condition of moisture content and density is seldom found to exist at the time the load test must be performed, so it is necessary to prepare a subgrade at the condition desired and then test it for bearing capacity. In North Carolina the soil to be tested for bearing capacity is brought to the laboratory and there prepared for the load test. The soil is weighed out in batches and mixed with the proper amount of water for the particular soil in a special mixer. The mixture is then placed in a bin 42 inches wide, 30 inches deep, and 14 feet long and compacted with pneumatic tampers to the required density, which is 98 to 100 percent of AASHO. The compacted soil is protected from moisture loss by covering the surface with heavy building paper, which remains on the soil until the test is completed.

The dimensions of the bin permit loading the soil in four 42-inch-square, identical sections, and the loading is done through four circular steel plates having diameters, 6⅔, 8, 10, and 13⅓ inches. Loads are applied in increments with a 50,000-lb. Black and Decker loadometer equipped with calibrated gauges for measuring the loads. The loadometer is attached to a beam, which is bolted to two columns situated near the ends of the bin. Settlements are measured to 0.001 of an inch with four micrometer dials, diametrically opposed. Figures 1 and 2 show the testing rig in operation.

The load testing procedure and analysis of data follows the technique developed by W. S. Housel and reported by him first, in 1936 (2), and later in 1941 (3). This method for determining the bearing capacity of soils to be used as subgrades or base courses is somewhat laborious and requires some time to perform, but the results are quite dependable, and it is felt that the end justifies the means.

Load tests are not made on the subgrade soils of every project designed, as the method used and time required to perform the tests make it impractical. The values of bearing capacity obtained from the tests made in the laboratory are used to rate the soils that will be encountered as subgrades. Soil or soil-aggregate mixtures are also tested as subgrades in order to determine their bearing capacity, which is their ultimate strength as a base course.

Table 2 appearing in the original paper presented in 1946 is presented here as Table 1. Values of subgrade bearing capacity obtained in our load testing to date do not warrant changing the values in this table. It might be mentioned that load tests made on soil type base course materials as subgrades give values slightly under 100 psi. Tests made on soil-aggregate mixtures give bearing capacity values well in

TABLE 1

TENTATIVE SUBGRADE BEARING VALUES

Subgrade Bearing Value	Description of Soil
30 psi.	Sandy soils, containing not more than 35 percent material passing a No. 200 sieve. Good drainage. Subgrade should be below the frost line.* Thickness of stratum must not be less than 12 in. Good Subgrade
20 psi.	Clay soils, but not heavy clay soils (50 percent or more clay). Good drainage. The subgrade must be below the frost line.* Fair Subgrade
15 psi.	Heavy clay soils or clay soils where drainage is uncertain. Subgrade should be below frost line.* Poor Subgrade
10 psi.	Heavy clay soils with poor drainage. Very Poor Subgrade

*Frost line is the depth below which water in the underlying soil will not freeze.

excess of 100 psi. This information is valuable when designing base courses for heavy axle loads whose pressures are 100 psi.

When rating a subgrade for bearing capacity, consideration is given to all factors affecting this characteristic, such as soil type, classification, drainage, and climatic conditions that prevail in that locality.

The distribution of pressure through flexible pavements is a moot subject. Most authorities agree as to the existence of such a phenomenon but do not agree on the form of distribution and, consequently, the amount of pressure reaching the underlying subgrade. Some authorities hold that the amount of distribution varies with materials and with the rigidity of the subgrade support, and others assume that the distribution takes the form of a truncated cone, the sides of which make 45 deg. with the vertical.

Roland Vokac has developed a concept based on the well-known pressure-bulb principle. This thesis appears in the Proceedings of the Asphalt Paving Technologists and the Proceedings of the Highway Research Board (4, 5). He assumes that the bulbs of pressures are spherical in shape, for mathematical simplicity, and reasons that the points on their circumferences made by the intersections of their diameters are points of zero vertical velocity and, therefore, points on the pressure-distribution curves. All forces within the bounds of the curve are downward and outward and all outside their boundaries are upward and outward. The direction of the forces at their intersection with the curves are horizontal only.

This concept and assumptions permit

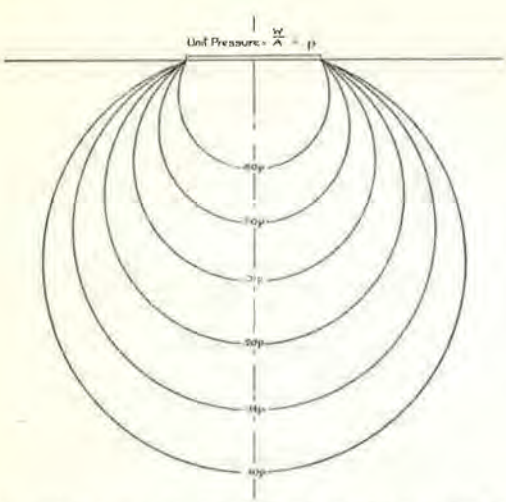


Figure 3. The spherical pressure bulb.

derivation of simple formulas for the calculation of average pressures on any plane below a loaded flexible surface, their diameters, and distance below the surface. This concept is shown graphically in Figures 3 to 5 and the derivation of the formulas in Figure 6.

Tables 2 to 8 give the thicknesses of flexible pavement required to carry axle loads of 8,000 to 20,000 lb. over subgrades having bearing capacities of 10, 15, 20, and 30 psi. using the formulas referred to above. In each table such information as contact area, equivalent diameter, and contact pressure are given for each load. The pressure given for each load is derived from data obtained from truck-weight surveys by increasing the average of the inflation pressures measured in the tires carrying a particular load by 10 percent for tire-wall stiffness. These tables are as they appear in the paper presented in 1946, "Current Base Design Practices in North Carolina", except that 1 inch (thickness of wearing surface) is added to the base thicknesses given in the original tables to give the total pavement thickness appearing in these tables. As mentioned at the beginning of this paper, the thicknesses given in these tables have been increased for axle loads above 13,000 lb. The method used to obtain these increased thicknesses will be explained in some detail later in this paper.

TABLE 2

RECOMMENDED FLEXIBLE PAVEMENT THICKNESSES FOR VARIOUS AXLE LOADS

Type of Load: Light single-unit trucks, medium loaded-
Axle Loads not exceeding 8,000 lb.

Wheel Load = 4,000 lb. Air Pressure + 10 % = 81 psi.
Contact Area = 49 sq. in. Equivalent Diameter = 7.9 in.

Subgrade Bearing	30 psi.	20 psi.	15 psi.	10 psi.
Total thickness of pavement in inches	5	7	8½	10½

Figure 7 shows the details of the calculations used to determine the thickness of a flexible pavement necessary to carry 20,000-lb. axle loads over a subgrade having a bearing capacity of 20 psi., using the method outlined above. The unit pressure exerted on the pavement surface is 100 psi. and is the inflation pressure carried by the tires, increased 10 percent for wall stiffness. The contact area, 100 sq. in., is the result of dividing the wheel load, 10,000 lb., by the pressure, 100 psi. The diameter of a circle having an area of 100 sq. in. is 11.3 inches. Substituting these values in the basic formula for pavement thickness we get 11.3 inches. It will be noted that Table 8 gives a thickness of 11½ inches, which is this value written to the nearest half of an inch.

TABLE 3

Type of Load: Light single-unit trucks, heavily loaded-
Axle loads not exceeding 10,000 lb.

Wheel Load = 5,000 lb. Air Pressure + 10% = 81 psi.
Contact Area = 62 sq. in. Equivalent Diameter = 8.9 in.

Subgrade Bearing	30 psi.	20 psi.	15 psi.	10 psi.
Total thickness of pavement in inches	6	8	9½	12

As stated before, the above method gives values that have proved inadequate for axle loads in excess of 13,000 lb., and has been revised for axle loads above this figure. The method has been given here because it is still used for axle loads of

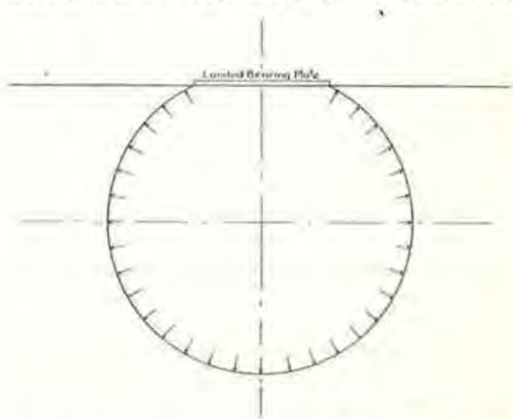


Figure 4. A finite bulb of pressure showing direction of forces.

13,000 lb. and under, and although it was described in the original paper, "Current Base Design Practices in North Carolina", it is repeated here in more detail in order to better describe the revised method.

REVISED METHOD

Séveral years after the above method of flexible-pavement design had been in use, it was noticed that one road which had been designed for 18,000-lb. axle loads was showing an excessive amount of failures. A study of the nature of these failures revealed that they were subgrade failures, as it was observed that settlement of the pavement was accompanied by upheaval of the adjacent pavement and shoulder. Since a subgrade failure can lead to only one conclusion, inadequate pavement thickness, the writer began to study the method of design to determine why the thicknesses were inadequate and devise a rational method for their increase, if possible.

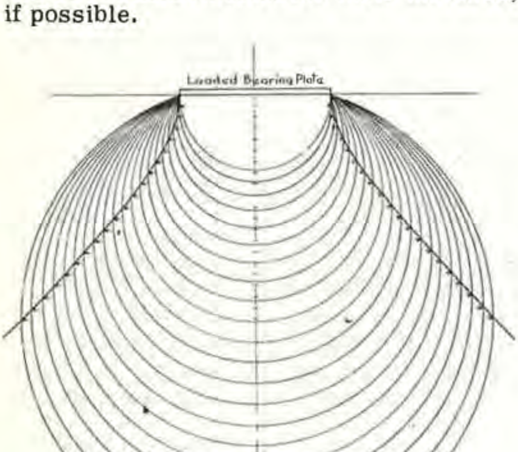


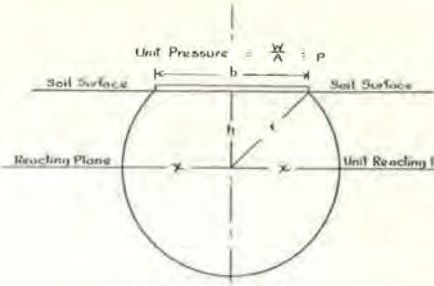
Figure 5. Points of zero vertical velocity within a pressure bulb.

TABLE 4

Type of Load: Medium single-unit trucks, medium loaded-Axle loads not exceeding 12,000 lb.

Wheel Load = 6,000 lb. Air Pressure + 10% = 83 psi.
Contact Area = 72 sq. in. Equivalent Diameter = 9.6 in.

Subgrade Bearing	30 psi.	20 psi.	15 psi.	10 psi.
Total Thickness of pavement in inches	6½	8½	10½	13



$$x^2 = r^2 = h^2 + \left(\frac{b}{2}\right)^2$$

Equating load on bearing plate to stress on reacting plane at depth h.

$$p\left(\frac{\pi b^2}{4}\right) = p_o(\pi x^2)$$

$$p\left(\frac{b^2}{4}\right) = p_o x^2$$

$$= p_o\left[h^2 + \left(\frac{b}{2}\right)^2\right]$$

$$p = p_o\left[\left(\frac{2h}{b}\right)^2 + 1\right]$$

$$h = \frac{b}{2}\sqrt{\frac{p}{p_o} - 1}$$

$$p_o = \frac{p}{\left(\frac{2h}{b}\right) + 1}$$

When Bearing Value of Subgrade equals p_o, the thickness of pavement necessary for unit load p is h inches.

Figure 6. Geometry of pressure bulb and derivation of stress formulas.

In the original method of design, the contact area was obtained from the load-pressure quotient from which the contact diameter was calculated. Since dual wheels have two areas of contact instead of one, it was decided to approach the problem from this angle.

Since the size of tires is related to load-carrying capacity, the larger tires carrying the heavier loads, it was necessary to obtain the width of tread and width between the tires on dual wheels for tires carrying the loads to be used in pavement design. This data was obtained at several of the permanent truck-weighing stations now operating in North Carolina and, after study, the values given in Table 9 were adopted for use in pavement design.

Figure 8 is a graphical illustration of the thickness design of a flexible pavement for 20,000-lb. axle loads when the subgrade bearing capacity is 20 psi. A 20,000-lb. load to be carried on one axle equipped with two dual wheels will require that the

TABLE 5

Type of Load: Medium single-unit trucks, heavily loaded-Axle loads not exceeding 14,000 lb.

Wheel Load = 7,000 lb. Air Pressure + 10% = 89 psi.
Contact Area = 79 sq. in. Equivalent Diameter = 10 in.

Subgrade Bearing	30 psi.	20 psi.	15 psi.	10 psi.
Total thickness of pavement in inches	7	9½	11	14

TABLE 6

Type of Load: Trailer-truck combinations, medium loaded-Axle loads not exceeding 16,000 lb.

Wheel Load = 8,000 lb. Air Pressure + 10% = 93 psi.
Contact Area = 87 sq. in. Equivalent Diameter = 10.5 in.

Subgrade Bearing	30 psi.	20 psi.	15 psi.	10 psi.
Total thickness of pavement in inches	7½	10	12	15

load carrying capacity of each of the four tires be 5,000 lb. Tires of this capacity belong to the 11.00 series which, according to Table 9 have treads 8 inches wide. When placed on dual wheel assemblies, the distance between treads was found to average 5 inches, according to the same table.

In the problem illustrated in Figure 8, the 10,000-lb. wheel load is applied to the pavement on two diameters of contact, 8 inches each spaced 5 inches apart, it being assumed, for simplicity, that the shapes of the contact areas are circles. The pressure exerted by the load carried by these two circular areas is 98.9 psi. and, as shown in the illustration, the pressure distribution lines intersect at a point 5.12 inches below the surface of contact. The pressure at this point is calculated to be 37.5 psi., but just below this point, due to the overlap of these stresses, the pressure becomes twice 37.5 psi. or 75 psi. The stressed plane at this point consists of two circular areas adjacent to one another and having diameters of 13 inches each.

By taking one of these circular areas, 13 inches in diameter and the pressure, 75 psi., and calculating the distance h_1 necessary to dissipate this pressure to 20 psi., the value 10.79 inches is obtained. Adding the value of h obtained above to this value of h_1, we obtain 5.12 + 10.79 = 15.91 inches of pavement required. It will be noted that only 11.3 inches of pavement is required using the original method of design (see Fig. 7).

Calculations of thicknesses required for axle loads of 8,000, 10,000, 12,000, 13,000, 14,000, 16,000, 18,000, and 20,000 lb. for subgrades of 10, 15, 20, 25, and 30 psi. of bearing capacity were made using the method described above. Tire tread widths and spacing for tires in dual assemblies designed to carry each load were used in accordance with the dimensions given in Table 9. The pavement thicknesses calculated in this manner were plotted against their respective axle loads and the points connected with smooth curves. Figure 9 shows these curves as solid lines. The dotted lines are pavement thicknesses calculated using the original method and are shown for comparison.

TABLE 7

Type of Load: Trailer-truck combinations, loaded to capacity-Axle loads not exceeding 18,000 lb.

Wheel Load = 9,000 lb. Air Pressure + 10% = 96 psi.
Contact Area = 94 sq. in. Equivalent Diameter = 10.9 in.

Subgrade Bearing	30 psi.	20 psi.	15 psi.	10 psi.
Total thickness of pavement in inches	8	10½	12½	16

Figure 10 shows curves for pavement thicknesses designed for eight axle loadings on subgrade bearing capacities of 10 to 30 psi. These design curves are used in North Carolina at this time with apparent success. It will be noted that a gap appears between the curves for the 13,000- and

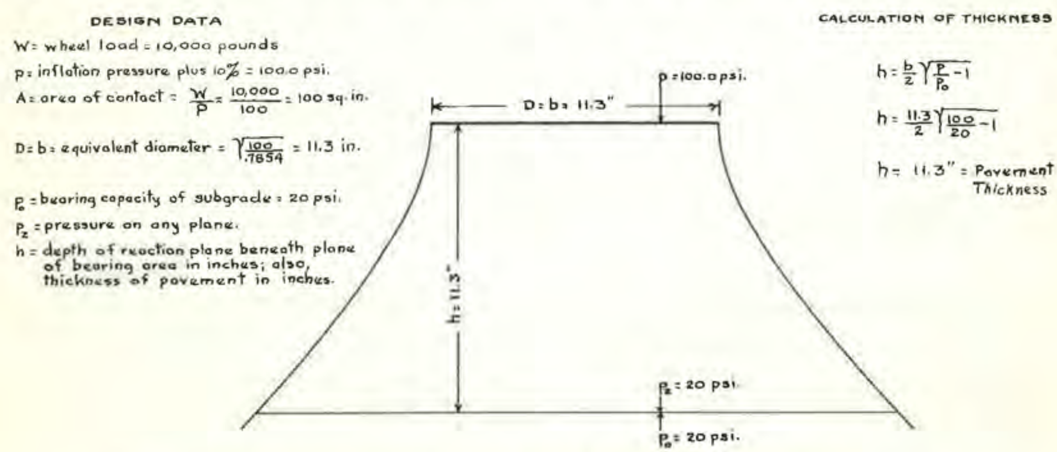


Figure 7. Flexible-pavement design, old method; axle load, 20,000 lb.

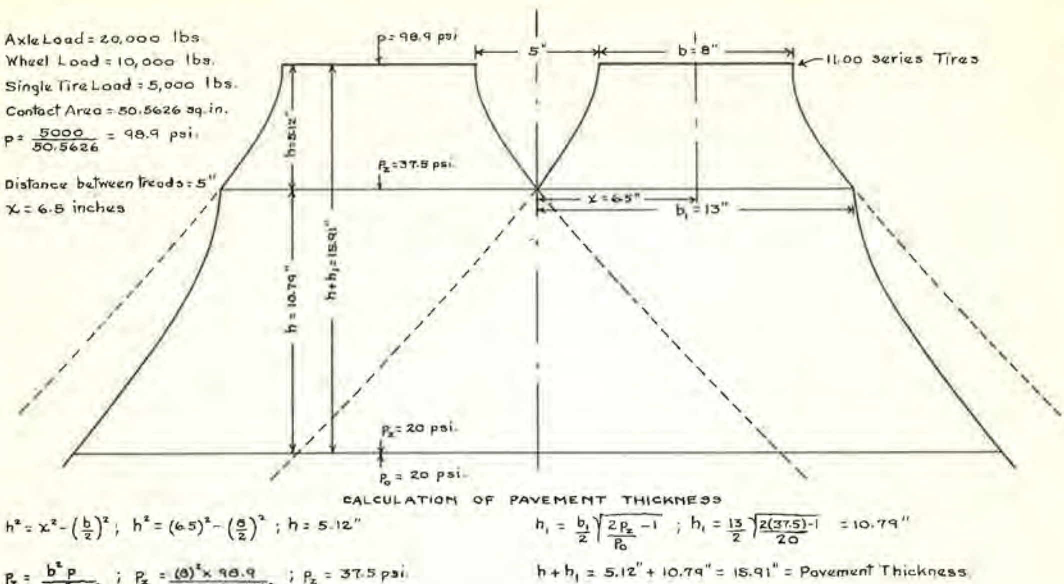


Figure 8. Flexible-pavement design, revised method; axle load, 20,000 lb.

14,000-lb. axle loads. This is due to the fact that the curves for axle loads of 13,000 lb. and under are plots of pavement thicknesses calculated by the original method, while the curves for axle loads of 14,000 to 20,000 lb. are for thicknesses calculated by the revised method.

Since pavement thicknesses designed by the original method have proved adequate for axle loads of 13,000 lb. and under, there is no reason to use the greater thicknesses given by the revised method, even though the revised method is considered more correct. This is an inconsistency that requires an explanation, and the author will try to do so in the following paragraphs.

In order to cause failure of a flexible pavement, by overloading it, an appreciable number of vehicles of this character are necessary. If traffic data is analyzed, it will be noticed that the general trend is for the weight of traffic to increase with the volume. Roads carrying heavy vehicles generally show high traffic counts, while those carrying lighter vehicles show lower counts. As an illustration, US 29 between Charlotte and Concord, North Carolina, has a traffic count of over 9,000 vehicles per day. Of this number approximately 2,300 are trucks of all sizes. A breakdown of the truck count shows over 1,000 of them to be the heavy type, tractor-trailers and combinations. The pavement on this road was designed for 20,000-lb. axle loads, using the revised design method which requires 16 inches of pavement on the type of subgrade (20 psi.) encountered. If the original design method had been used, only 11½ inches would have been required, which thickness has proved inadequate for loads of this magnitude on this type of subgrade.

TABLE 8

Type of Load: Trailer-truck combinations, heavily loaded-Axle loads not exceeding 20,000 lb.

Wheel Load = 10,000 lb. Air Pressure + 10% = 100 psi.
Contact Area = 100 sq. in. Equivalent Diameter = 11.3 in.

Subgrade Bearing	30 psi.	20 psi.	15 psi.	10 psi.
Total thickness of pavement in inches	9	11½	13½	17

In contrast with this, a county road in northern Granville County has a traffic count of 460 vehicles, of which only 90 vehicles are trucks. These trucks are all of the pickup type and school busses. The pavement on this road was designed for 8,000-lb. axle loads, using the original design method which requires 7 inches of pavement on the type of subgrade (20 psi.) encountered. This thickness has proved adequate for this road. If the revised design method had been used, 10 inches of pavement would have been required.

In the first instance, the Charlotte-Concord road, if the truck count had been 90 heavy trucks per day instead of 1,000, the thickness given by the original design method, 11½ inches, would have proved adequate; and in the second instance, the county road in Granville County, if 1,000 school busses per day should use the road,

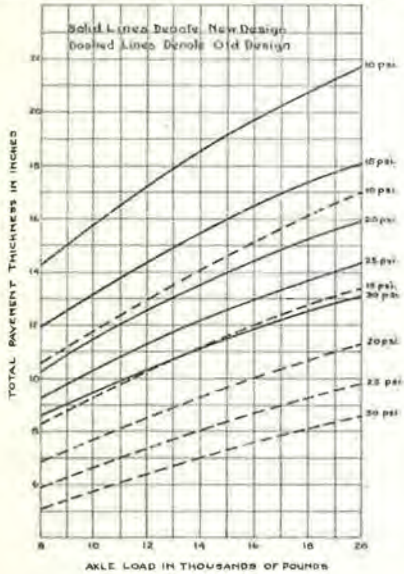


Figure 9.

the existing 7-inch pavement given by the original design method would in all probability fail. The 10-inch thickness required by the revised design method probably would be adequate.

The author feels that the above reasoning is logical and, in view of the fact that the two design methods give thicknesses that are proving adequate, believes that the use of two methods, although inconsistent, is justified. A rational method of design for the thickness of flexible pavements has been used in North Carolina since 1945, and the above methods are the result of much careful research, investigation, and study. The use of Vokac's development of pressure distribution based on the pressure-bulb principle is believed to be the most-logical and applicable one offered at this time, and until important research work now in progress by a large agency is complete and reported, its use will be continued.

TABLE 9

TIRE SIZES, THEIR DIMENSIONS, AND MANUFACTURERS' RATED CAPACITY USED IN FLEXIBLE PAVEMENT DESIGN CALCULATIONS

Tire Series	Mfg. Rated Cap.	Tread Width	Dual Wheel Tread Space
	lb.	in.	in.
7.50	2,000-3,000	6	4.5
8.25	2,700-3,000	6	4.5
9.00	3,500-4,000	7	5
10.00	4,000-4,500	7.5	5
11.00	4,500-5,000	8	5

SELECTION OF BASE AND SURFACE TYPE

This paper, up to this point, has dealt with the thickness design of a flexible pavement. Although it has been mentioned that the components of the pavement structure (subbase, base, and surface) must have bearing capacities at least equal to the pressures applied to them, no mention has been made of the type of materials suitable to use in their construction. The following is a discussion of the merits and demerits of the various materials used in North Carolina and recommendations as to the types most suited to specific purposes.

The pavement structure rests on a prepared subgrade which, we will assume, has a bearing capacity sufficient to support the pavement and the loads that use it. The pavement structure may consist of a subbase, base, and surface course, or it may consist of just a base and surface course.

TABLE 10

SOIL TYPE BASE COURSE
(Fine Aggregate Type)

Soil Type Base Course material of the fine aggregate type shall not contain more than 35% of aggregate passing the 2-inch and retained on the No. 10 sieve, and its soil mortar fraction (material passing the No. 10 sieve) shall conform to the following grading requirements when tested in accordance with AASHO Method T88:

Sieve Designation	Percentage by Weight Passing
No. 10	100
No. 40	40-75
No. 200	12-35

The fraction passing the No. 200 sieve shall be less than two thirds of the fraction passing the No. 40 sieve. The material passing the No. 40 sieve shall have a plasticity index not greater than 6 and a liquid limit not greater than 25, when tested in accordance with AASHO Methods T89, T90, and T91.

Surface Courses

Since the surface course must receive the action of traffic directly, it must possess certain inherent characteristics that will permit it to function properly. In the case of flexible pavements, it must be flexible enough to deform and recover under traffic loads, yet stable enough not to be distorted. It must be watertight so as not to permit water to enter its mass and cause its deterioration or pass through it and soften the base beneath. It must also possess toughness to resist the abrasive action of traffic and durability to resist the action of the elements.

Bituminous surface treatments are very-thin wearing courses constructed by apply-

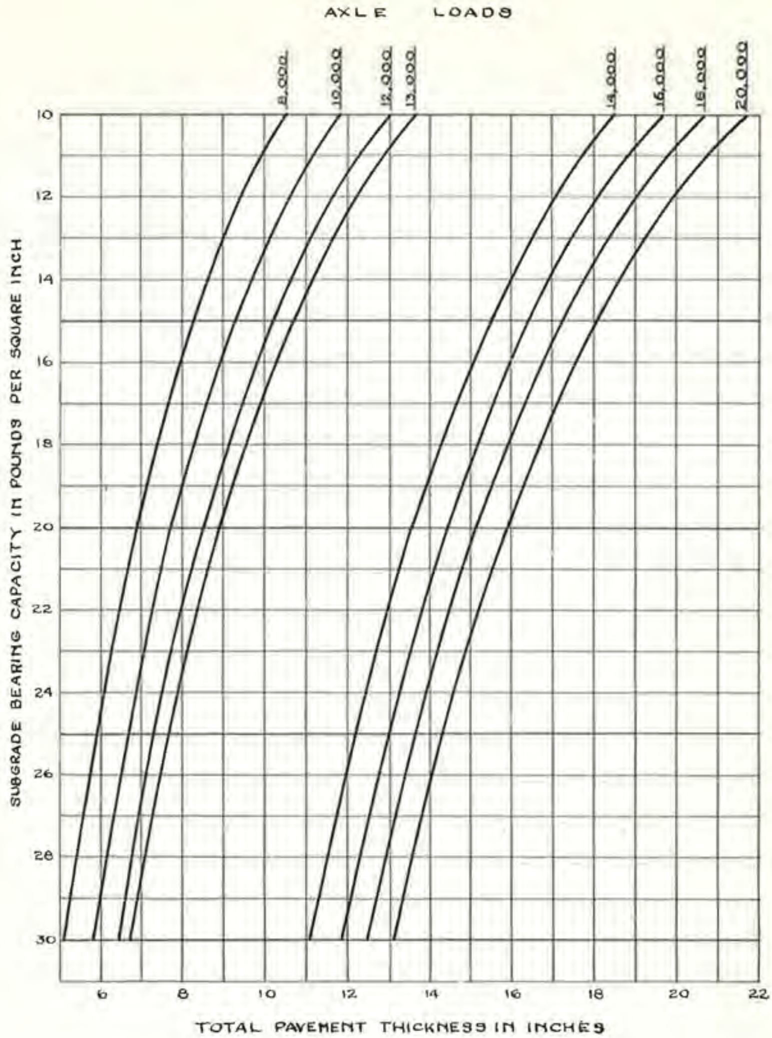


Figure 10. Flexible-pavement design curves.

ing a liquid bituminous material to a base course and covering it with aggregate. The bituminous material acts as a waterproof membrane to protect the base course from surface water and as a cohesive substance to hold the aggregate. The aggregate serves as an armor coat to resist the abrasive action of traffic. This is an excellent surface course for light- to medium-weight traffic, but it must be placed on suitable base courses, as its lack of thickness does not permit any appreciable reduction in pressure reaching the base course due to load distribution. Pavements designed for axle loads of 13,000 lb. and under may consist of soil type bases and this type of surface course, but when the design loads are greater than this figure, an aggregate type base is recommended.

Bituminous surface treatments may be used as temporary surface courses, to be followed by plant-mix surfaces later, on aggregate-type base courses designed for heavy axle loads; however, they should be carefully maintained. Breaks that develop should be sealed or patched immediately to prevent the entrance of water into the base and eventually into the subgrade. Where the traffic is heavy in volume as well as weight, the author prefers the use of the higher-type surface course, or part of it, in the beginning. If part of a plant-mix surface course is used, it should either be placed to a thickness sufficient to be impermeable or else consist of a very-closed

TABLE 11

SOIL TYPE BASE COURSE
(Coarse Aggregate Type)

Soil Type Base Course material of the coarse aggregate type shall contain at least 35 percent of aggregate retained on the No. 10 sieve, and shall conform to the following grading requirements when tested in accordance with AASHO Method T88:

Sieve Designation	Percentage by Weight Passing
2-inch	100
1-inch	70-100
½-inch	55-100
No. 4	40-80
No. 10	30-65
No. 40	15-45
No. 200	5-25

The fraction passing the No. 200 sieve shall be less than two thirds of the fraction passing the No. 40 sieve. The material passing the No. 40 sieve shall have a plasticity index not greater than 6 and a liquid limit not greater than 25, when tested in accordance with Methods of the AASHO T89, T90, and T91.

mixture, such as sheet asphalt.

Plant-mix bituminous surface courses are considered to be the highest type of flexible-pavement surfaces. When properly designed and constructed, they are flexible yet stable, are impermeable, and durable. Manufacture of the mixture can be controlled to the degree that the designed mix will meet rigid specifications, and modern methods and machinery make it possible to place the material in such a manner that a smooth, well-compacted surface is obtained. Where local materials are available and stability requirements not so high, a comparatively cheap surface course may be constructed of plant-mix material. The thickness of these surface courses may be as low as 1 inch. The use of this quality of surface course is, of course, confined to pavements used by light traffic.

Bituminous surface courses that are to be used in the construction of pavements designed to carry heavy loads should be of the plant mix type, and the mixture should be carefully designed for stability and its preparation and placement controlled. The mixture should be rolled to a high degree of compaction and the surface course thickness should not be less than 3 inches. Placement in as many thin layers as practicable is good construction practice, as this procedure will produce a higher degree of compaction and a smoother riding surface than would otherwise be obtained.

Base Courses

A base course is that part of the pavement structure on which the surface course rests. It suffers no abrasive action, but may be required to withstand high pressures if surfaced with a thin surface course. It must be well bonded and consolidated or it will be distorted, if not destroyed, by the action of traffic.

Soils have the property of taking up a certain amount of moisture, although protected by an impervious cover and kept well above any water table, especially in the spring (1). Granular type soils take up less moisture than soils of the silt-clay type and are affected to a lesser degree by its presence. The presence of coarse material in soils increase their stability materially, and it is possible to design specifications for soils and soil-aggregate mixtures that will permit their selection or production for base course work. Tables 10, 11, and 12 give the North Carolina specifications for three base-course materials.

The specification given in Table 10 is for a soil-type base material containing little or no coarse aggregate. Bases constructed of this type of material will function satisfactorily with bituminous-surface-treatment surfaces or 1-inch plant-mix surfaces when the axle loads are not over 13,000 lb. The bearing capacity of this type of material is generally less than 100 psi., and its use beneath surface courses less than 3 inches thick on roads designed for heavy axle loads is not recommended. This material may be used in the construction of a subbase beneath 6 inches of coarse-aggregate-type base course to produce a combination that will have the strength or bearing capacity of a coarse-aggregate base of the same thickness as the combination.

Tables 11 and 12 contain specifications for coarse-aggregate-base materials. The specification in Table 11 was designed to suit coarse aggregate type soil materials that occur in natural deposits or that may be produced by mixing two or more materials obtained from natural deposits. The bearing capacity of materials meeting the requirements of this specification is above 100 psi.

The specification in Table 12 is for artificial mixtures of coarse aggregate and fine material. It is generally produced of quarry materials, crushed stone and stone screenings, but may be produced by combining any suitable coarse aggregate with soil of the proper grading and quality. This material is required to be mixed in a pugmill-type mixer with sufficient water to prevent segregation and for proper compaction and is spread on the subgrade or subbase with approved mechanical spreaders in 4-inch loose layers. The material is then immediately compacted to a density of not less than 100 percent of the density produced by the AASHO compaction test as modified by the U. S. Engineer Department. Calcium chloride is added to the mixture during mixing operations at the rate of 7 lb. of calcium chloride to the ton of mixture for use in the top 3 inches of the base course. On certain projects the calcium chloride is required to be mixed with all of the material.

TABLE 12

STABILIZED AGGREGATE BASE COURSE

The coarse aggregate fraction (material retained on the No. 4 sieve) shall consist of crushed stone, crushed or uncrushed gravel, crushed air-cooled blast furnace slag, or other inert materials having similar characteristics, unless otherwise specified. The slag shall weigh, when dry and rodded, not less than 70 lb. per cu. ft., when tested in accordance with AASHO Method T 19. The mixture of the coarse aggregate and fines when analyzed prior to spreading on the road, shall meet the grading requirements, using AASHO Method T88, as follows:

Sieve Designation	Percentage by Weight Passing
2-inch	100
1-inch	75-95
½-inch	60-75
No. 4	45-60
No. 40	15-30

No. 200 (Not less than 5 percent nor more than two thirds of the percentage passing No. 40 sieve.)

The material retained on the No. 4 sieve shall consist of tough, durable pieces of aggregate which, when tested in accordance with AASHO Method T 96, Test Grading A, will show a loss not greater than 45 percent. When subjected to 5 alternations of the soundness test, AASHO Method T 104, using sodium sulfate, the weighted average loss shall not be more than 15 percent.

The material passing the No. 4 sieve shall be known as "fines", and shall consist of screenings produced by the crushing of acceptable rock, or other material of satisfactory bonding value which the Laboratory may approve.

After the base course has been completed, that portion of the material which passes the No. 40 sieve shall have a plasticity index not greater than 6 and a liquid limit not greater than 25, when tested in accordance with AASHO Method T 89, T 90 and T 91.

The materials shall be mixed in an approved mechanical mixer. Water shall be added during the mixing operation in an amount designated by the Soils Laboratory, which will not be less than 5% nor more than 9%.

This is considered to be the highest type of base course used in North Carolina, and since greater control can be maintained of the materials and their placement than any other type, it is preferred in the construction of flexible pavements designed for heavy loads. Approximately 130 miles of this type of base course have been constructed in North Carolina during the past 2 years, and the pavements, of which they are a part, are serving some of the heaviest traffic in the state without distress. It is planned to use this type of base course in the future in the construction of all flexible pavements designed for heavy loads, where practicable. This type of base course is especially suited to the widening of old existing pavements, which comprises a large part of the highway construction program in North Carolina at this time.

Subbase Courses

A subbase, if used, is that part of a pavement structure on which the base course rests. It has the same function as the base course and should possess the same characteristics, except that it need not possess as high a bearing capacity, as its position in the pavement structure does not require that it withstand as much pressure. For this reason less coarse material may be permitted, but it should meet all of the other requirements for soil type base material given in Table 10. The use of a subbase in the design of a flexible-pavement structure, especially if such materials are available locally, permits a substantial saving in the cost of the pavement.

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Flexible-Pavement Design by the Group-Index Method

W. C. DAVIS, Chief, and W. G. JONES, Senior Engineer,
Geology and Soils Branch, Division of Materials,
Missouri State Highway Department

●A METHOD of design, based mainly on group index and heavy traffic, has been used by the Missouri State Highway Department since 1947 to determine total thickness to recommend for flexible pavements.

Prior to that time the principal distinction between projects within the secondary system was established by the traffic count. The base of a lightly traveled road usually consisted of 4 to 6 inches of stabilized clay-aggregate mixture, while 6 inches of higher-type base material (such as waterbound macadam, soil cement, or bituminous stabilized material) was generally used on roads carrying heavier traffic. Interpretation of the exact meaning of lightly and heavily traveled roads was pretty much left up to the judgment of the designer. Little consideration was given to base thicknesses greater than 6 inches, because of a belief that subgrade soil so deficient in stability that it required more than 6 inches was too weak to support a flexible pavement. Therefore, in such cases, the routine course was to design for concrete.

Total thickness varied due to the practice of varying the bituminous wearing course, which might be an armor coat, double seal, oil mat, or asphaltic concrete.

Through the years following completion, these roads showed great differences in performance. Many functioned well until heavier type traffic and the unavoidable decrease in maintenance during the war years began to exact their toll.

The end of World War II marked the beginning of a change in our policy regarding soils information which, prior to that time, had been given to the designer without definite recommendations concerning its most-advantageous use. Since few designers were equipped with the specialized knowledge necessary to make full use of the information provided, there was some reluctance even to try to use it. Fundamentally, design was based on past experience in Missouri.

The somewhat-gradual deterioration noted in many of our roads during the war years was speeded up by the marked postwar increase in traffic volume and weight. As a result, the spotlight was directed toward our rather casual attitude concerning the importance of base thickness, emphasizing the inadequacy of our design and indicating the need for the development and adoption of a definite design method. Additional stimulus was provided by the return of many engineers from war work, where they had assimilated ideas concerning the latest practices in design and construction of flexible pavements, as well as in the practical application of soils data obtained from field and laboratory investigations.

However, "the homecoming of the wise men" could hardly be called an unmixed blessing, inasmuch as discordant opinions were frequently held and strongly defended by designers, construction men, and soils engineers. The all-important straw was added to the camel's load in 1947 when the Soils Division specifically recommended an 8-inch base in an area where 6 inches was considered by the designer to be ample thickness.

This recommendation engendered much discussion but the base thickness was set up for 6 inches. Perhaps the most-important result of all the conversations was that one idea crystallized from many, when it was decided that some definite method of thickness design should be established for use by the Missouri State Highway Department.

It was realized that the group index was developed as a soil-classification tool, easily determined from simple soil tests performed by all laboratories. We were aware that bearing values, obtained in one or another of several ways, were used by numerous organizations to determine base thickness, and that we might be among the minority if we chose to ignore the path followed by so many.

For several reasons, however, we elected to do just that, not the least of which, perhaps, was the show-me attitude of the Missourian. Nobody had convinced us that any one bearing test was better than any other, and our routine tests did not include those for either bearing value or shear. We felt that, since the group index involved no tests complex in themselves or in the diagnosis of their results,

The proposed chart was preferably to be uncomplicated and to utilize only such soil tests as were commonly made in our laboratory. Although at the time of origination it was considered to be only the first step in the right direction, the second step has not been taken and the chart in use today remains essentially the same as the original.

The chart was developed in the following manner:

1. CBR and Proctor density tests were made on all horizons of 32 typical soils of

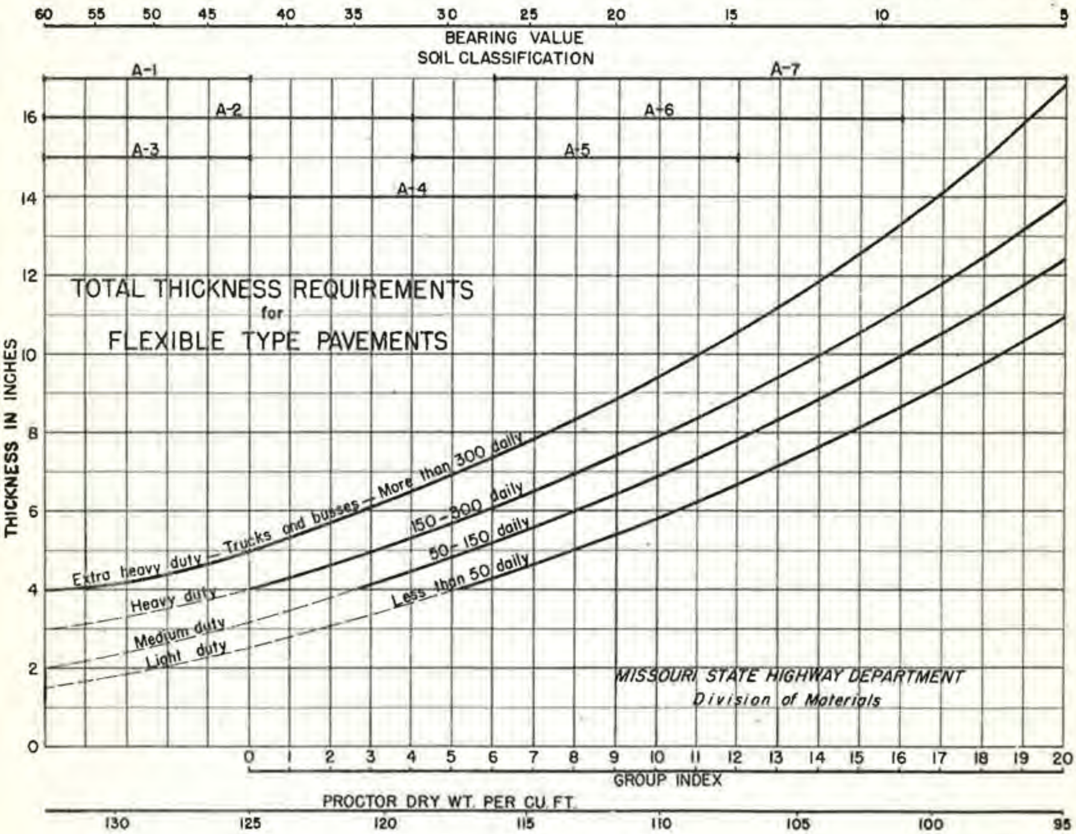


Figure 1.

it met our demands for simplicity and should serve satisfactorily as the basis for our design method.

Various ideas were advanced and different approaches tried before the final decision was reached that our thickness chart would be based primarily on the group index of the subgrade soil and the volume of heavy traffic (trucks and busses), with minor adjustments being influenced by the maximum dry weight of the soil.

Missouri, with the exception of most of the thin, organic A horizons.

2. CBR tests were correlated with bearing value as shown in sketch, SCX-16 assembled by the Portland Cement Association (1).

3. Use was made of Highway Design Charts H-10, H-15, and H-20, presented in an Asphalt Institute Progress Report (2) to determine the correlation between varying bearing values and total thicknesses of

base and asphalt surface. These values were slightly changed before plotting, with the extra-heavy-duty curve corresponding to the Asphalt Institute Curve H-20, heavy duty corresponding to H-15, and medium duty corresponding to H-10. The light-duty curve was added by extrapolation.

4. It was thought that repitition of critical loading was more important than wheel loads alone. With some changes, D. J. Steele's (3) designation of light, medium, and heavy traffic was applied. Our classification agreed with his in the light duty class (less than 50 commercial vehicles daily), but because a different set of conditions prevailed, we called his "heavy duty" extra-heavy duty, (300+) and broke down his "medium duty" into medium duty and heavy duty (50-150 and 150-300).

5. From the Portland Cement Association sketch the relationship between CBR, bearing value, and PRA (now BPR) classification was established. The BPR classification as shown in the sketch was, however, changed to be in line with the 1947 proposed revision (4).

6. Charting the revised BPR classification, which included maximum group index limits for each A group, automatically established the maximums on our thickness chart, while the minimums were fixed to conform to experience with Missouri soils.

The thickness chart is used by the Plans and Surveys Bureau, since it is that bureau's responsibility to set up all phases of the design, including thickness and width, type of base, and surfacing. However, furnishing reliable information concerning soils, excavation classification and drainage is one of the functions of the Division of Geology and Soils. The field representative of this branch of the department assumes the responsibility for obtaining the necessary information, interpreting soil test results and field conditions, and offering suggestions that will help the designer to derive greatest benefit from the available data. He finds use of the chart essential to complete fulfillment of these obligations.

On projects through new locations where grading work must be done, the district geologist or soils engineer makes borings to determine the presence of any ledge stone and to classify the different soil types according to the U. S. Department of Agriculture, Bureau of Soils, Method. Natural-density tests are made for estimating soil shrinkage, and samples from each horizon of each soil type are submitted to our Jefferson City Laboratory for routine tests.

The geologist writes a soil-survey report, which includes a description and test data for each soil involved in grading operations or which will form subgrade for the base, notations about any unusual drainage conditions, shrinkage factors for use in computing earth balances, location of ledge stone if it occurs, and other geological features. This report, with the geologist's interpretation of all the pertinent information obtained from both laboratory and field work, is submitted to the designer.

On reconstructed routes where no grading is required, frequent and accurately measured soundings determine the salvable thickness of the old wearing course and base, and borings are made to determine the major soil changes in the subgrade. Typical samples of subgrade material are tested. Where base sufficient to be of value in stage construction is found, it is sampled and tested for P. I. and gradation. A report is then submitted to the designer describing the amount of salvable material and its probable value, the subgrade soils, and drainage conditions.

After evaluating the effects of the group index and the latest traffic count of heavy vehicles, the designer uses the thickness chart as a guide to determine the total thickness of base and surfacing. Consideration of a material survey report on available materials enables him to specify the type of base and surfacing to be used. Information contained in the soil-survey report helps him to specify the type and location of special drains, and such selective soil handling as may be necessary or desirable.

Application of the thickness chart usually involves only group index and heavy traffic; but in certain cases, as in the following example, consideration is given to the maximum dry weight. Assuming a group index of 10 and extra-heavy traffic (300+) the chart indicates the necessity of a total thickness of 9. 5 inches. Whether to specify 9 inches or 10 inches is then influenced by the dry weight. Since the group index of 10 almost exactly coincides with 110 lb. per cu. ft., a weight greater than 110 would be considered to be a better soil and need less total thickness. Theoretically, then, 9 inches would be specified. Thus, a lower weight, poorer-type soil would demand a greater thickness, and 10 inches would ordinarily be specified.

Since its adoption, the chart has been used to design thicknesses for projects in various parts of the state which, because of the scattered locations, offer contrasting soil, climate, drainage, and traffic conditions. Detailed descriptions of the design, construction, and performance of some of the projects follow:

ROUTE SK, ANDREW COUNTY

This 5.25-mile project extends from Amazonia to the Buchanan County line at the edge of St. Joseph, in northwest Missouri. About a third of this project lies in the Missouri River flood-plain area adjacent to a prominent escarpment, while the remainder of the project is located in heavily rolling upland topography. This contract consisted of a few short sections of re-alignment, raising the grade in some of the bottomland area, and constructing a rolled-stone base with bituminous surface. Construction was started in April and completed in November, 1949.

During a soil survey in the spring of 1947, the limits of the soils were determined, and those that would comprise the subgrade for the new base were sampled and tested. The soils were identified as Knox and Marshall silt loams in the upland area, and Sarpy clay loam, Pennsylvanian shale, and glacial till in the low area adjoining the escarpment. Fifteen representative soil samples showed group indexes ranging from 2 to 11. Based on a medium-duty traffic count and these group indexes, the total thickness recommendations by the division geologist for the designer's guidance were 9 in. in the first 2,550 ft., 5 in., 7 in. or 9 in. in the next 8,700 ft. (depending on the source of borrow material for raising the grade), and 7 in. in the remaining 16,615 ft. The suggestion was made that French-type shoulder drains be installed through areas of the more-impervious subgrade.

A material survey, made in this locality in 1944, indicated no gravel available in the immediate area. Two limestone sources near the project were located and sampled, both deposits being identified as the Oread formation of Pennsylvanian age. Portions of the ledge were accepted for use in the production of aggregate for bituminous surfacing and the entire sampled thickness was accepted for use as rolled stone base.

Changes in plans subsequent to the original soil survey necessitated variations in thickness from those recommended. The thickness of the 24-ft.-width base as constructed was 9 inches for the first 1,510 feet; 7 inches for 5,140 feet; 9 inches for 440 feet; 7 inches for 1,060 feet; 9 inches for 2,030 feet; and 7 inches for the remaining 17,685 feet, or a total of nearly 24,000 feet of 7-inch base and approximately 4,000 feet of 9-inch base. The 7-inch base was constructed in two lifts, and the 9-inch base in three equal lifts. The density specified was at least 95 percent of the standard Proctor maximum. The bituminous surface course was constructed to a width of 22 feet, and consisted of two applications of MC-5 with a total of about 45 lb. of aggregate per square yard.

The daily traffic count on this route in 1948 just before it was closed for new construction was 901 vehicles, of which approximately 150 were trucks and busses, thus making it a borderline case between medium and heavy duty. The medium-duty classification was used as a basis of design, even though past experience indicated that some increase in traffic is a logical expectation after improvement of a project. The 1952 traffic count on this project followed the usual pattern, increasing to 1,360 vehicles, of which about 240 were trucks and busses. This indicates that anticipated traffic should have been the basis of design, in which case the heavy-duty curve would apply and a greater base thickness would have been specified.

Maintenance costs for a 13-year period before reconstruction averaged $355 per mile per year. Since reconstruction the maintenance costs have averaged $249 per mile per year for the 3-year period, of which more than 92 percent was spent for cover material when the bituminous surface showed excessive bleeding shortly after construction was completed. Repairing small failures accounted for the remainder.

The condition of this route in October of 1953 was unsatisfactory. The first 1.7 miles of the project, which is on the flat grade through the bottom, and hence, in an area of poor drainage, showed somewhat more severe distress than the remaining 3.55 miles located in the hills. Through the 1.7-mile section there were eight small failed areas totaling about 700 sq. ft. Several other areas showed severe cracking and distortion. Much longitudinal cracking and some corrugating was particularly noticeable in the outer wheel tracks. In the 3.55-mile section, there was one failure of approximately 300 sq. ft., a few other areas were severely cracked and distorted, and longitudinal cracks with slight accompanying depression appeared in most of the cuts and on the low sides of curves.

An attempt will be made to remedy the existing condition by resurfacing next spring with a plant mix of penetration asphalt and mineral aggregate sufficient to give an additional thickness of about 3.5 inches.

ROUTE 59, ATCHISON COUNTY

This north Missouri project extends from the Iowa line to Tarkio, a distance of 7.8 miles over lightly rolling topography through soils of loessial and glacial origin. The old road was oiled earth. New construction, between August 1949 and July 1950, included a few sections of light grading but primarily consisted of placing a rolled-stone base and bituminous surface on the oiled earth.

A soil survey made in December of 1948 showed that the subgrade generally consisted of one of the three horizons of the Marshall silt loam, a loessial soil, although in some areas a glacial soil (Shelby silt loam) was found. Five representative soil samples were tested, three showing a group index of 8 and two of 11. Based on a medium-duty traffic count and the group indexes, the chart indicated that thicknesses should be 6 and 7.3 inches, so a total thickness of 7 inches was recommended throughout the project. No special drainage was recommended.

Material surveys several years previous to 1949 had indicated no stone deposits of any consequence in the county and only small scattered deposits of poor quality glacial sand and gravel. Therefore, crushed stone was shipped from the most-economic source, a distance of approximately 65 miles.

A rolled-stone base 6 inches thick and 22 feet wide was constructed throughout the project. The lower of two lifts was 4 inches thick, the upper was 2 inches and the specified compaction for each was at least 95 percent of Proctor maximum density. The base was topped with a double bituminous surface treatment 20 feet wide, in which the total aggregate used amounted to about 60 lb. per sq. yd.

The average daily traffic count in 1949 before construction started was 493 vehicles, of which approximately 69 were trucks and busses, placing it near the lower limit of our medium-duty classification (50-150). In 1952, 2 years after completion of the project, the traffic showed a very slight increase to 526 vehicles, of which about 73 were trucks and busses.

The maintenance costs for 13 years previous to reconstruction averaged $470 per mile per year. Since reconstruction the annual per mile surface maintenance costs have averaged $152. Maintenance has consisted of repairing a few failures and spot sealing surface blemishes.

A condition survey of the project in October 1953 showed one large failure of about 500 sq. ft. and three other failures totaling 50 sq. ft. Another 2,000 sq. ft. has shoved and corrugated, and longitudinal cracks are almost continuous throughout the project in the outer wheel lanes.

Improvement similar to that for the previously described route is scheduled for this project in 1954, when approximately a 3.5-inch-plant-mix will be placed.

ROUTE STT, ST. LOUIS COUNTY

This project is a 4,600-foot relocation in the northern part of St. Louis County, in east central missouri, through rather level terrain along a terrace or second bottom of Coldwater Creek. Grading of the new location started in August of 1951, and the construction of a rolled-stone base surfaced with asphaltic concrete was completed in October, 1952.

From the soil survey made in December of 1949, it was determined that the project was located in an area of reworked loessial soils. These soils were identified as Bremer silt loam, a soil containing a large amount of silt and organic matter, and Robertsville silt loam of similar texture but containing much-less organic material. Samples representative of the horizons of each soil type were tested. Results showed almost identical group indexes of 8, 10, and 13 for the A, B, C horizons of the Bremer and 8, 10, and 14 for the Robertsville.

An effort to anticipate the worst condition influenced the division geologist to suggest in his soil-survey report that the group index of 14 be used as the criterion for determining the pavement thickness. Based on this group index and an anticipated extra-heavy traffic count, the recommendation was made that 12 inches of total thickness be used throughout the project. Special drainage installations were not necessary, and the materials problem was simplified because several established quarries operated nearby.

The recommendations for this project were followed without variation. A total pavement thickness of 12 inches was constructed, consisting of a 9-inch rolled-stone base, 26 feet wide, and a 3-inch asphaltic-concrete surface, 24 feet wide. The base was compacted in three equal courses, each to at least 95 percent of Proctor maximum density.

The daily traffic count on this project in 1950 was 761, of which about 33 were trucks and busses. Relatively little traffic used the route before improvement, because of the narrow, hazardous roadway. Recent data is not available, but observation indicates that the increase has been considerable.

The condition of this road at present is excellent and there have been no maintenance costs.

ROUTE 21 TR, JEFFERSON COUNTY

This project is a relocation of a portion of Route 21 bypassing the City of DeSoto. The 6.84-mile project traverses hilly topography of the Ozark region, where deep cuts and high fills involved excavating and handling large quantities of both ledge stone and dirt. Grading started in June of 1947, and the pavement, consisting of rolled-stone base with a bituminous seal, was completed in July of 1948.

The original soil survey for this route was made in 1940, when the soil type throughout the project was identified as Union silt loam, a soil which is usually composed of fine-grained material of loessial origin overlying cherty, granular, residual material. Group indexes of samples taken at this time varied from 10 to 14.

The project was designed in 1946, which was previous to the development of our thickness chart. The designer, with little information to guide him, set up the project for a 6-inch base with a bituminous seal. Excavation by the grading contractor exposed such extensive, heavy clay subgrade that construction personnel questioned the adequacy of the design and requested that a new soil survey be made.

The resurvey in 1947 showed nearly all fills to be capped with clay and the subgrade through most cuts to consist of shattered dolomite. Eight soil samples, considered to be representative of clayey sections throughout the project, showed group indexes ranging from 11 to 16. These figures, considered in conjunction with anticipated traffic in the medium-duty classification, indicated the necessary total thickness to be from about 7.5 inches to 10 inches. A supplemental report was submitted recommending a 9-inch base, which would be slightly increased by addition of the bituminous seal and probably fall just a trifle shy of the maximum thickness indicated by the chart to be necessary through the worst sections. A minimum of 4 inches of base material was recommended in the rock cut sections.

Adoption of these recommendations meant that the project had to be divided into clayey and stony subgrade sections varying in length from 550 feet to 3,040 feet and involving 19 changes of base thickness. The only special drainage installation was made to intercept intermittent seepage from a wet weather spring in the subgrade.

A material survey in the locality showed an abundance of Jefferson City dolomite of Ordovician age which was suitable for use as base material. Stone suitable for bituminous surface could be produced by selective quarrying of certain ledges. Local gravel deposits were insufficient in size to supply the necessary quantity of base material. Sufficient gravel of suitable quality for use as bituminous surface material was available in nearby stream deposits.

With but few exceptions, a 9-inch rolled-stone base was constructed in the locations recommended. A 6-inch base was built in all stony subgrade sections instead of 4 inches as suggested. The base throughout the job was constructed in 3-inch layers, each of which was compacted to at least 95 percent of Proctor maximum density prior to placement of the next course. The base was full uniform thickness for 20 feet and feathered out on 8-ft. shoulders. Prime was applied to the full 36 feet, and a bituminous surface consisting of 75 pounds of stone chips per sq. yd. was placed on the 20-foot width.

Since this project is on new location, traffic comparisons before and after are not possible. The first available traffic count was in 1952, which showed a daily average of 1,133 vehicles (about 240 trucks and busses). In 1953 this traffic had increased to 1,246 vehicles, of which 262 were of heavy classification. Both of these counts are well up into the heavy-duty class instead of the medium-duty for which the project was designed.

The exact maintenance costs on this 6.84-mile section are unavailable. Prorating the 4-year costs of a maintenance section which includes this project and an adjoining 5.5-mile section of similar construction gives an average of $129 per mile per year.

Within a year after completion of this project two areas failed. Both were on fills and grades, and both occurred where the base thickness was 6 inches. The smaller (160 sq. ft.) showed a marked stratification in the subgrade material with several inches of silty topsoil overlying tight clay. The capping layer picked up moisture which could not escape and developed a condition which resulted in failure. The larger failure (2,250 sq. ft.) occurred on clay subgrade and was probably caused by one of the few unaccountable variations from the recommended procedure. A 9-inch base on this section would undoubtedly have reduced the probability of failure. The same statement might be made concerning the smaller failed area, but better construction procedure would have spread the topsoil on ditch inslopes or mixed it with the clay, rather than concentrate it in a layer capping an impervious clayey soil.

Accurate observations of this pavement have been infrequent since 1949, but conversations with maintenance patrolmen indicate that no further failures have occurred. Minor surface blemishes have developed in the traveled way and raveling is conspicuous on the primed shoulders.

In 1952, this project was resurfaced with 3 inches of asphaltic concrete for a width of 22 feet as a protective surface coating, rather than as insurance against the development of base failures. No surface maintenance has been charged against the job since that time and the road is in excellent condition.

ROUTE 62, NEW MADRID COUNTY

This project extends from Risco to Malden, an almost perfectly flat distance of 8.2 miles across part of an old Mississippi River flood plain near the Bootheel section of southeast Missouri.

The 20-foot pavement, previous to improvement, consisted of concrete slabs, some of which were 9 feet and some 10 feet wide. The remaining 10 or 11 feet consisted of either bituminous surfaced thin gravel or oiled-sand base.

Although the concrete slabs were performing satisfactorily, the bituminous widening presented a constant and expensive maintenance problem. Through the years various methods of treatment were used in an attempt to improve this road. It was felt that some salvage value was recoverable from the existing roadway so the design was set up to make full use of such material as was available.

A condition survey made in February of 1948 to determine the quantity and quality of salvable material showed wide variations in base material. Fairly clean and rather dirty sand was found, as were oil-sand mixtures, clayey gravel, and clean gravel in amounts that were not at all consistent but were revealing as to the attempts to build stability into the job in past years. Estimates were made of the amount of recoverable material in the various sections.

The alluvial subgrade soils were identified as Sharkey and Waverly types, varying from organic clay loams to sandy loams. The group indexes of the eight samples tested varied from 0 to 12.

With the expectation that the road would be subjected to traffic corresponding to our extra-heavy classification, recommendations were made for total thicknesses varying from 6 to 11 inches as the quality of the subgrade soil varied. Twelve changes in thickness were involved in sections of lengths varying from 530 feet to 10,725 feet. The report also contained a recommendation that water be removed from the base by drains through the shoulders, even though dissipation of water collected by ditches in this area is usually accomplished by the slow process of evaporation. Normal conditions in this level section of Missouri usually mean high humidity and high water table, neither of which are conducive to rapid elimination of surface water.

Material deposits are scarce in this section of Missouri, being confined primarily to rounded chert and quartz gravel and sand deposits containing varying amounts of binder and occurring in a long hill known as Crowley's Ridge. Suitable deposits within this ridge had been previously located at a distance of 7 or 8 miles from the project.

Construction of this job was completed between August 24 and November 25, 1948. With the exception of 9,000 feet of 6-inch base near the middle of the section, the base thickness throughout was 8 inches. Base material was produced by two methods: The lower portion consists of a mixture of broken, salvaged, bituminous material and any underlying base of whatever nature, while the top course was stabilized gravel in sufficient quantity to produce the specified thickness.

Because the gravel in its natural state contained too little fine material and showed too high plasticity to meet our specification, fine sand (also a Crowley Ridge deposit) was hauled to the project and windrowed in quantity such as to bring the total material within specification limits on both counts. The sand, gravel, and moisture for compaction were mixed in one operation by means of a traveling mixer. Minimum compaction was specified as 95 percent of Proctor maximum density. The base was placed in a trench 11 or 12 feet wide, as determined by the slab width, and was surfaced with an open-type, coarse-graded, asphaltic concrete, 2 inches thick and 1 foot narrower than the underlying base. Recommendations were ignored and no drainage of any type was provided.

The contractor was working against time and time was fast running out on him, as it often does for anyone proposing to place asphaltic concrete during November in Missouri. Unfortunately, several rains fell while long sections of trench were lying open. Removing excess moisture from wet subgrade by aerating and discing is a fine idea in certain circumstances; but in damp, cool, cloudy weather over a southeast Missouri flat, undrained, trench section it was a lot of waste motion. A few soft and unstable spots were removed, but many were not, and the base was apparently mudded-in in quite a few areas.

The failure of this road might be, for the most part, ascribed to the fact that construction was hurried because the season was so late. In order to reduce the probability of a winter shutdown on an unfinished job, good construction practice was disregarded and the effects soon became evident.

The daily traffic count in 1947, a year before reconstruction of this route, was 1,146, of which about 225 were trucks and busses. By 1953 these figures had increased to 1,369 vehicles, of which 280 are of the heavy type. Into this category fall many trucks often thought and sometimes proved to be loaded considerably in excess of the legal maximum of 18,000 lb. per axle. Watermelons, soybeans, and cottonseed are produced in large quantities, and in the fall huge loads are often hauled over roads inadequately designed to withstand such weight.

The average annual surface maintenance cost of this section for an 11-year period prior to reconstruction was \$360 per mile. Since reconstruction (1949 through 1952) the cost has averaged \$392 per mile per year. These figures include the costs for resealing the entire section and repairing broken and failed areas.

As indicated by these figures and confirmed by visual observation, the reconstruction of this road was not successful. About 75 small, scattered failures totaling approximately 100 sq. yd. developed and were repaired by the contractor before he left the job. Failures subsequently grew in number and size and other defects appeared, until by the following summer (1949) 7 percent of the total area of new construction showed complete failure, 8 percent was severely depressed or near failure, 6 percent was slightly depressed, and 33 percent had been resealed to cover the cracks. Most of the distressed areas occurred in the western 2 miles, which were the latest constructed, and nearly all failures were observed in areas over the poorer subgrade soils.

Serious deterioration shortly thereafter came to almost a complete standstill, with new failures developing nowhere except within the 2-mile section, where the occurrence of new defects proceeded at a much slower pace. At present this pavement is performing satisfactorily, except for the western 2 miles which, although rough, is still serviceable. No additional construction is planned for this project at the present time.

IMPROVEMENT OF SECONDARY ROADS

Many secondary roads in Missouri have been built up over a period of years, primarily as a result of routine maintenance. Periodic addition of replacement material, ditch pulling, maintenance, stabilization and maintenance oil mats, decks, seal coats, etc., are all instrumental in building thickness, stability, and some value into such roads. In conjunction with a thickness survey, our chart has been used as a guide when some of these roads have been scheduled for further improvement, as well as on projects too short to be included in this report. Missouri has recently done a great deal of widening and resurfacing of old, narrow and defective concrete pavement, where the chart has been used in determining the thickness of the widened portion on many of these projects.

Other instances of its use, none of which have any place in this report, may be cited as on projects originally set up for flexible pavements but on which the final design was for concrete. There are also a few projects in various stages prior to letting of the contract, on which chart recommendations have been reported and which may or may not reach the construction stage as flexible pavements.

Since its introduction as a tool for design, unforeseen obstacles have sometimes barred the way to as complete use of the chart as was expected, and it has, in some cases, been used improperly or not at all, experiences which are probably not peculiar solely to Missouri.

We feel that we have accumulated insufficient data to permit an accurate evaluation of our method of design at the present time. However we cannot fail to recognize the fact that it falls short of perfection, if there is such a thing in flexible-pavement design. We in Missouri have just about abandoned hope of providing a design method that will solve all problems in all conditions and revolutionize the controversial subject of flexible-pavement-thickness design. If we can develop one that works satisfactorily in Missouri for Missouri conditions, we will consider that we have reached one of our important objectives.

Factors in Method Improvement

We believe that the accumulation of more data, both on completed projects and those yet to be constructed, will bring into sharper focus the shortcomings of our design method which we hope to improve by incorporating the knowledge derived from the study of several phases of the general road building program, in which we expect to emphasize the importance of the following factors:

Anticipated Traffic

More-accurate forecasts of the traffic to be expected in the future would permit greater accuracy in our thickness recommendations. The possibility of underdesigning and thereby repeating the experience illustrated by Route K, Andrew County, and Route 21, Jefferson County, would be greatly reduced. As described previously, these roads were designed for medium-duty traffic, but improvement attracted traffic in such amount that they are well within our heavy-duty classification.

Overdesign of thickness in anticipation of traffic volume that never materializes is not good design practice, but might be justified by a saving in maintenance costs greater than the cost of the additional material.

Type of Base and Surface Course

Our method does not take into consideration the difference in bearing value between the various types of bases and bituminous surfaces. Crushed stone, keyed together by its angularity and lightly bound by the slight cementing action of its dust, probably has greater stability than a well-graded, rounded gravel containing soil binder. We know there is a difference in bearing strength between lime, soil-cement, stone, and mechanically stabilized bases, but our design does not take this into consideration.

Frost and Drainage

It is about 300 miles from the Iowa line at the north to the southeastern tip of the Bootheel in Missouri, which is only 70 miles from the Mississippi state line. Average frost penetration varies from about 20 inches in north Missouri to about 5 inches in the Bootheel. The detrimental effect of frost on pavements in Missouri is not great but may be sufficient to warrant consideration in any revision of our method of design.

Good design procedure must necessarily provide for removal of water which collects in a base. If drains are not installed, additional thickness might help to compensate for the lack. In our design we make no allowance for additional thickness if the drainage is poor.

Construction

Latent though fundamental assumptions underlying our method of design are: (1) construction materials meet our specifications; (2) field testing and inspection for gradation, plasticity, moisture, density, etc., is accurate and test results are reliable; and (3) construction methods comply with those accepted as standard practice.

When one or more of those assumptions proves to be not well founded and failures appear in the road, we hope that further experience will provide the knowledge that will enable us to pinpoint the causes of trouble.

Proposed Group-Index Revision

We have done some work and intend to do more toward revising the method of determining the group index. We propose an approach to the problem which will enable us to reduce the amount of testing necessary, basing the determination on P.I. and plastic limit. Perhaps adaptation of the agronomists' field capacity and wilting point, in terms of P.I. and plastic limit, will enable us to determine the group-index limits from a different viewpoint than at present. We believe there should be some method of taking into account the decreasing stability inherent in a soil while its moisture content ascends the plastic range.

The members of the Soils Division of the Missouri Highway Department recognize at least some of the inadequacies of our present method. Since the failures occurring on some of the projects designed by our method can easily be accounted for by factors other than erroneous soils information, we have had no particular reason to consider re-examination, evaluation, and revision of our design method. When that point is reached, we believe that complex tests will not necessarily provide the complete answer to our problems.

A systematic study of conditions before, during, and after construction and accurate, periodic observations of the performance of various type flexible pavements, in soil and climatic conditions as found in Missouri would, we believe, more nearly provide the information we now lack for making an intelligent revision of our design method. Such a program has not been inaugurated in Missouri, but we have hopes of starting such a study in the near future.

Additional impetus has been given to a review of our method of design by a recent directive to the effect that, on secondary roads in the Missouri system, a minimum 8-inch base and minimum $1\frac{1}{2}$ inches of bituminous surface will be constructed, unless a lesser thickness is justified by the soil survey. Since this provision, if always adhered to, would eliminate from consideration approximately 70 percent of our chart, a critical analysis of the many and various methods of design becomes the first order of business in Missouri. An effort is being made to determine whether something better than our present method can be developed, retaining if possible the simplicity which we feel is one of the desirable characteristics in the group-index method of design.

REFERENCES

1. "Interrelationships of Soil Classifications, California Bearing Ratio, Bearing Values, and K Values." Feburary 1944.

2. Progress Report No. 1 - "Design of Asphalt Pavements for Highway and Air-

port Engineers" by the Engineering and Development Committee, Division 3. Copyright 1945.

3. Discussion by D. J. Steele in 1945 HRB Proceedings - "Application of the Classifications and Group Index in Estimating Desirable Subbase and Total Pavement Thicknesses."

4. Highway Research Board Tabulation "Changes and Proposed Changes From 1931 Public Roads Soils Classification System". January, 1947.

Modified C.B.R. Flexible-Pavement Design

D. J. OLINGER, Principal Materials Engineer,
Wyoming Highway Department

Approximately 1,450 miles of road have been constructed during the past 7 years following the modified California Bearing Ratio flexible-pavement design initiated in 1946. This method is reported in the 1947 PROCEEDINGS of the Highway Research Board as "Wyoming Method of Flexible-Pavement Design."

The modified C.B.R. controls (1) overall thickness of pavement, base, subbase, and that portion of the basement soil classed as selected embankment and (2) the thicknesses and quality of the selected embankment and subbase.

The base and pavement have had no test requirements for stability, other than the Wyoming stability test, so they have been designed both for thicknesses and quality by past experience plus gradation, liquid limit, plasticity index, and wear tests. A test procedure is now being put into practice to allow a better selection of base and pavement aggregates.

Included are tabulations of: (1) roadway and pavement widths, also surfacing widths, based on traffic and state highway system; (2) traffic variations within each state highway system; (3) several near typical highway projects showing number of soil samples taken, number of different designed thicknesses, etc., for both the preliminary and construction stages; and (4) the opinions of the various engineers of the Wyoming Highway Department as to the merits and weaknesses of the present design.

DURING the past 7 years the modified C.B.R. flexible pavement design has been used in bringing to completion, including a bituminous pavement, approximately 350 miles of reconstruction and 1,100 miles of new construction on Wyoming's 4,800-mile state highway system. Many more miles are in various stages of construction, but not entirely completed.

The present design procedure was initiated in 1946, and reported in the 1947 PROCEEDINGS of the Highway Research Board as "Wyoming Method of Flexible Pavement Design." Briefly, this method includes: (1) compacting the C.B.R. specimen under static load at optimum moisture to the standard AASHO density; (2) soaking the specimen four days with a ten pound surcharge; and (3) using the bearing value determined on a design curve selected by an empirical evaluation of (a) annual precipitation, (b) depth of water table below profile grade, (c) frost action, (d) general existing conditions, and (e) traffic in order to determine the total overall thickness of pavement, base, subbase, and selected embankment.

The modified C.B.R. is used in determining the quality, also the thickness that may be used, of selected embankment and subbase materials. However, the base and pavement have had only the Wyoming stability test, gradation, liquid limit, plasticity index, wear test, and past experience used as the criteria for design thickness and quality of materials.

MODIFICATIONS OF METHOD

Modifications have been made from the original method when revision was justified, as follows:

1. The material originally called "imported fill" has been reclassified as "selected embankment."

2. Thicknesses of pavement and base are held uniform on any one job, and the thicknesses of subbase or selected embankment are used as the adjustable components. These have been modified to vary in uniform increments of 2 or 3 inches, instead of the ½-inch increments originally indicated. This reduces the number of sections necessary on a project, and simplifies the staking.

3. Selected material surfacing is now crushed to a 2-inch maximum in place of allowing up to 4-inch-maximum size in a 6-inch lift. This allows traffic a better riding surface on stage construction.

4. The original minimum center sections, (shown in Figure 1 of the original

TABLE 1

ROADWAY WIDTHS, PAVEMENT WIDTHS, AND SURFACING THICKNESSES, BASED ON TRAFFIC

	TRAFFIC - ESTIMATED EQUIVALENT 5000 POUND WHEEL LOADS FOR 20 YEARS IN ONE DIRECTION - IN MILLIONS													
	0-0.25	0.25-1	1-3	3-5	5-7	7-9	9-11	11-14	14-17	17-21	21-26	26-32	32-40	40-50
Width at bottom of base	26'	28'	30'	40'	40'	40'	40'	42'	42'	42'	42'	48'	48'	48'
Width of bituminous pavement	20'	22'	24'	28'	28'	28'	28'	28'	28'	28'	28'	28'	28'	28'
Minimum total thickness	4½"	5"	5½"	6"	6½"	7"	7½"	8"	8½"	9"	9½"	11"	11½"	12"
Minimum pavement thickness	1½"	1½"	1½"	2"	2"	2"	2"	2½"	2½"	2½"	2½"	2½"	2½"	2½"
Minimum base thickness	3"	3"	3"	4"	4"	4"	4"	4"	4"	4"	4"	5"	5"	5"
	FEDERAL AID SECONDARY			FEDERAL AID PRIMARY				FEDERAL AID INTERSTATE						
	STATE HIGHWAY SYSTEM													

paper) have been increased as indicated in Table 1. The 1½-inch pavement is road mix, the 2-inch pavement either road or plant mix, and the 2½-inch pavement is plant mix laid in two courses. This change was based upon (1) a large increase in equivalent 5,000-lb. wheel loads revising the 1951 to 1971 traffic on some roads up to over 40 million in one direction, whereas the 1946 to 1966 traffic had indicated a high of approximately 15 million, and (2) the conclusion that the original minimum thicknesses were inadequate. These data are shown in Table 2.

5. Occasionally, where conditions are unfavorable, it is the practice to increase total thicknesses above the minimum indicated by the design procedure. This practice is most often used in low areas with high water table, and may double the total minimum required thickness.

SERVICE

Generally the pavement and its substructure components are giving satisfactory service, but in a small percentage (less than 5 percent) of total constructed mileage the performance has not been satisfactory. Briefly, where investigations of failures have been made, poor service has been attributed to one or more of the following:

1. Preliminary soil survey; (a) samples not representative; (b) not recognizing, or properly evaluating, special problems; (c) omission of pertinent data from the preliminary soil profile; (d) soil survey delayed so that testing and design is not allowed adequate time for proper handling.

2. Testing and design; (a) improper processing of samples; (b) not recognizing, or properly evaluating, special problems; (c) not reviewing the tentative preliminary design in the field with the project engineer.

3. Construction practices: (a) pockets, or sections, of unstable basement soil "bridged over" in lieu of subexcavation and backfill; (b) drainage problems not recognized and corrected; (c) no field adjustments in thicknesses made as based on experience and field construction control tests; (d) construction samples not submitted in time to allow revised design of construction thicknesses by the central laboratory; (e) insufficient control of materials.

IMPROVED DESIGN FOR BASE AND PAVEMENT

Experience has proved that (1) some granular materials satisfactory for pavement and base materials would just pass modified C.B.R. requirement for subbase, yet these same materials may have given satisfactory service as base and pavement for a number of years, and (2) some granular materials having a relatively high C.B.R. had inherent characteristics that did not contribute to satisfactory bases and pavements.

The materials section of the Colorado Highway Department had tested a number of the questionable granular materials for the Wyoming Highway Department with Hveem Stabilometer testing equipment, and the resulting design thicknesses were much closer to those indicated by experience than the modified C.B.R. design had indicated. Furthermore, the equipment appeared to be sensitive to small changes in gradation or plasticity, thus it would enable one to evaluate increased stability, if desired, by improving gradation, and addition of binder or filler.

Similar stabilometer equipment has been installed in the Wyoming laboratory for future use on granular materials, including the pavement. It is not expected that this will solve all problems associated with granular materials, only that it will give a more practical evaluation of many materials than has been obtained previously.

DESIGN OF NEAR TYPICAL PROJECTS

Near average projects for secondary, primary and interstate highway systems are shown in Table 3, with a breakdown of soil samples, granular materials samples, tests, and design, on both preliminary and construction. These projects are referred to as "near average" for they include only the middle ranges of soils, and exclude any projects that (1) fall mostly in the A-1-b(0) and A-2-4(0) classifications with modified C. B. R. high enough that thicknesses are governed mostly by the minimum total thickness requirements indicated in Table 1 or (2) fall mostly in the A-6(10 to 16) and A-7-6 (12 to 20) classifications with modified C. B. R. mostly low enough, (3.0 percent or less) that maximum total thicknesses are required for the particular design curve being used, plus any other additional thicknesses allowed for thru cut sections, or low areas with a water table relatively close to profile grade.

The project engineer makes the preliminary soil and materials survey. The samples are tested, and preliminary design set up, by the central laboratory. This design is usually reviewed in the field with the project engineer and adjustments made. Later, during construction, it may be necessary to again make revisions based on construction samples plus other revised data. The projects shown in Table 3 are typical examples of the near-average projects in Wyoming, and study of the tabulation will give an understanding of variations occurring within different projects.

QUESTIONNAIRE ON DESIGN

Recently a questionnaire regarding design methods and practices was submitted to all assistant project engineers, project engineers, maintenance engineers, and district engineers; 32 of 39 engineers considered that they had sufficient experience to answer one or more sections of the questionnaire, and their answers to the main points are given in Table 4.

Thicknesses of Materials

The engineers listing overall thicknesses as being inadequate qualified their statements by pointing out a specific section of a project, or a particular project, and these references were to a very small percent of the total mileage constructed by each engineer. Approximately 25 percent of the engineers answering felt that the 1½-inch road-mix pavements were inadequate, with few unfavorable comments on the 2-inch pavements, and none on the 2½-inch pavements. The engineers reporting base thicknesses as inadequate were referring to the 1½-inch and 2-inch center-base thicknesses used in the past, or the 3-inch-minimum center thicknesses now being used on secondary roads.

TABLE 2

TRAFFIC VARIATION WITHIN EACH WYOMING HIGHWAY SYSTEM

System		U. S. ROUTES	SECTION	TRAFFIC 1941 Daily Total	1941 Daily Commercial	1951 Daily Total	1951 Daily Commercial	20 Year Factor 1951 to 1971	1971 Est. Daily Total	1971 Est. Daily Commercial	20 Year Estimate of Equivalent 5000 lb. Wheel Loads in One Direction 1951 to 1971
Federal Aid	Interstate	30	Hanna-Sinclair	1096	147	2014	319	3.0	6042	957	40,525,435
		85 & 87	Cheyenne-South	1044	208	2750	451	3.0	8250	1353	27,780,428
		30	Rock Springs-Gr.River	1561	278	2517	412	2.7	6796	1112	28,266,299
		20	Evansville-Glenrock	1287	233	2698	350	3.0	8094	1050	25,620,263
		87	Kaycee-Buffalo	299	97	514	97	2.5	1285	243	3,109,976
		14	Moorcroft-Carlile	247	55	340	32	2.2	748	70	933,692
Federal Aid Primary		85	Newcastle-Mule Cr.Jct.	269	89	683	205	2.7	1844	554	13,908,143
		20	Moneta-Hiland	469	80	862	141	2.5	2155	353	6,713,212
		189	Big Piney-Daniel	295	92	190	32	2.2	418	70	788,499
Fed. Aid Secondary			Medicine Bow-Casper	----	---	56	10	*15.0	----	*150	*4,532,698
			Gillette-Broadus, Mont.	----	---	56	13	2.7	151	35	439,524
			Manderson-Hyattville	80	13	198	29	2.2	436	64	126,329

*Estimated upon completion of thru route.

TABLE 3

TABULATION OF SAMPLES, TESTS AND DESIGN THICKNESSES ON A NEAR AVERAGE PROJECT ON SECONDARY, PRIMARY AND INTERSTATE HIGHWAYS

PROJECT		PRELIMINARY											CONSTRUCTION			
		SAMPLES				TESTING		SURFACING DESIGN					SAMPLES	TESTS	DESIGN REVISED	
Number also Estimated Equiv. 5000 lb. Wheel Loads in One Direction in 20 Years in Millions	LENGTH	Soil	Soil Comparisons in Field	Granular Materials	Predominate Soil Types	Modified C.B.R.	Pavement Thickness	Base Center Thickness	Subbase Thickness	Sel. Embankment Thickness	Soil Lab. & Field	Granular Mat'ls. Lab. & Field	Modified C.B.R.	Increased Thickness	Decreased Thickness	Total
	Miles	No.	No.	No.	Classification	No.	In.	In.	In.	In.	No.	No.	No.	Miles	Miles	Miles
INTERSTATE I 221(6) 40.5	6.98	16	14	96[1]	A-2-4(0), A-4(2 to 8) A-6(8 to 9)	28	$2\frac{1}{2}$"[2]	5"[3]	0	4", & 10"	L.22 F.77	L. 32 F.140	0	2.22 2", & 3"	0.96 3", & 6"	3.18
							$11\frac{1}{2}$" Minimum Section									
PRIMARY F 76(2) 3.6	4.88	28	0	17	A-2-4(0), A-4(3 to 8), A-6(6 to 11)	10	$1\frac{1}{2}$"[4]	$3\frac{1}{2}$"[4]	4"	0", 3", & 9"	L.15 F.30	L.25 F.50	2[6]	0.20 2"	0.00	0.20
							9" Minimum Section[5]									
SECONDARY S 84(1) 0.4	8.59	11	11	19	A-3(0), A-2-4(0), A-4(2)	8	$1\frac{1}{2}$"	3"	4", & 6"	0	L.18[7] F.39[8]	L.34[7] F.54[8]	4	4.69 2"[9]	0.00	4.69
							$8\frac{1}{2}$" Minimum Section									

[1] An extensive granular materials survey was made on this project.
[2] $2\frac{1}{2}$" plant mix will be contracted at some future date.
[3] Top 2" of 1" base was oil stabilized with $4\frac{1}{2}$% of MC-3, as temporary pavement.
[4] Designed & constructed prior to latest revision in thicknesses, see Figure 1.
[5] Each project has a minimum section set up on preliminary that governs construction.
[6] Most construction samples have C.B.R. referenced to, or estimated from, a similar preliminary.
[7] Laboratory tests include weekly check tests, finals on the pavement and its substructure components, etc.
[8] Field tests include density tests, quality tests, etc., used in the field for control.
[9] 4.69 Miles of the 4" subbase was increased 2", to a total of 6", based on construction samples

TABLE 4

ANSWERS TO QUESTIONNAIRE ON MODIFIED C. B. R. FLEXIBLE-PAVEMENT DESIGN

	QUESTION	Inadequate	Adequate	Excessive	Yes	No	Not Answering
THICKNESS	Total thicknesses of pavement, base, subbase & sel.mat'l.surf. have been	3	26				3
	Pavement thicknesses (as shown in Fig. 1) have been	7	21				4
	Base thicknesses (as shown in Fig. 1) usually have been	5	22				5
	Subbase thicknesses usually have been	6	6				20
	6" additional (used occasionally) thru cuts on C.B.R. of 3.0-% is	2	18				12
QUALITY	Has lack of proper quality in pavement sometimes contributed to failure				14	11	7
	Has lack of proper quality in bases sometimes contributed to failure				15	10	7
	Has lack of proper quality in subbases sometimes contributed to failure				15	6	11
	Should the quality of the bituminous pavements be improved				27	0	5
	Should the quality of the bases be improved				25	1	6
	Should the quality of the subbases be improved				23	1	8
MISCELLANEOUS	For a balance between safety and drainage, which of the following is your choice for finish pavement crown per lineal foot of width of pavement (1/8"				8		2
	(3/16"				16		
	(1/4"				6		
	Which do you prefer (An untreated shoulder				1		
	(A bituminous treated shoulder				16		
	(Extension of pavement to the shoulder line				4		
	(Pavement to the shoulder line & slope treatment				11		
	(Other				0		
	Do you prefer the ditch section to extend well below any granular pavement substructure components				23	1	8

Note: Seven of the thirty-nine engineers did not feel that they had sufficient experience to answer any portion of the questionnaire.

Quality of Materials

Nearly all of the engineers indicated their opinion that the quality of the pavements, bases and subbases should be improved, mostly by better gradation or addition of binders and fillers, yet only about 50 percent indicated that lack of quality in these materials had contributed to failures

Miscellaneous Design Considerations

The present pavement crown of $\frac{1}{8}$ inch per lineal foot of width was considered inadequate for proper drainage by approximately 75 percent of the engineers, with 50 percent preferring a $\frac{3}{16}$-inch pavement crown, and 20 percent favoring a $\frac{1}{4}$-inch crown.

In regard to pavement width and shoulder construction, approximately 45 percent favored extension of the pavement to the shoulder line, and approximately 35 percent favored a slope treatment beyond this point. Those not favoring extension of the pavement to the shoulder line wanted a bituminous treated shoulder similar to the double shot bituminous treatment being used.

The standard ditch section drops $1\frac{1}{2}$ feet below profile grade (carried at the bottom of the base at shoulder), and the practice of deepening this ditch, where necessary to keep it well below any granular materials placed, was approved by most of the engineers familiar with this practice.

SUMMARY

It is estimated that over 95 percent of the mileage constructed by the modified C. B. R. design during the last 7 years is at present giving satisfactory service. The fact that trouble often becomes evident in very short periods makes one hesitant on future predictions of service without accurate projected traffic data and test track studies.

Designing Flexible Pavements (Virginia)

D. D. WOODSON, Assistant Testing Engineer,
Virginia Department of Highways

● THE design of flexible pavements in Virginia is based on the California bearing-ratio method (C. B. R.) with certain modifications. Since everyone interested in the subject is more or less familiar with this method, only the modifications will be elaborated on in this paper.

First it was decided that since the state specifications required that all fills and subgrades be compacted to 95 percent of theoretical density as determined by AASHO Method T-99, that it would be better to test the specimens at this density rather than at the density specified in the procedure as originally developed, as this would more nearly meet field conditions. Realizing that it would be impossible to obtain these densities with the procedure as set forth in the recognized method, it was decided that the test specimens be compacted using the 5½-lb. compaction rammer. The next thing that had to be determined was the number of blows necessary with this rammer to obtain the right density using the larger C. B. R. mold. After many tests with different soils, it was found that proper results could be obtained by using 45 blows and compacting the soil in five layers. The original test as developed required that clay soils be soaked until there was no further swell or expansion. Since several agencies had decided on a 4-day soaking period for all samples, it was decided that we would standardize on this time. All samples are now soaked for this period of time. Of these modifications the density at which the specimens are compacted seems to have the greatest bearing on the resultant design. This density naturally gives a lower C. B. R. value, which necessitates a greater thickness of treatment. It is felt, however, that since the specified density of the subgrade is the same as that at which the specimens are tested, the design depth is nearer to what it should be to carry the load than if the higher density was used.

Along with the values obtained from the modified test procedure, the anticipated traffic and loads are also used in the design depth of the pavement. A set of curves, as shown in Figure 1, based on those that were developed by the Corps of Engineers is used to determine the depth of pavement necessary for the various loadings. You will note also on Figure 1 that a minimum depth of pavement is required for given loads.

To explain the mechanics of the method, I believe a description of a project where this method was utilized would more clearly define the procedure.

As soon as the location survey was finished, the proposed alignment and grades were furnished to the soils laboratory, so the soil survey could begin. At this time a request was also made for a traffic study, so the anticipated traffic and loads could be determined. This information was also to be used in the final design. The soil survey was made with a mechanical soil auger capable of making borings up to 100 feet in depth. This enabled the survey crew which usually consists of an operator, two soil technicians, and two to three laborers to thoroughly explore the deepest cuts throughout the project. Usually the borings are made to 3 feet below the proposed grade. Samples were obtained for all types of soils encountered in both fill and cut sections. A complete log was kept of the changes in soil encountered throughout the project and recorded on profile plan sheets. Samples were also taken of the various layers of soils. After all of the test information had been obtained, this information was useful in selecting the better soils to be used for capping purposes during the construction of the project.

On this particular project which was on an old location and where little change in grades and alignment were contemplated seven samples were actually tested. The C. B. R. soaked values ranged from 3. 2 to 30. 0. Two of these samples were of the existing soil base and naturally had the higher bearing value. Careful consideration was given to all of the characteristics of the soil samples along with the C. B. R. values in order to determine which were the better soils to use in the upper layers.

There was one long section on the project where the soil was found that had the C. B. R. value of 3. 2. It lay directly beneath 'the existing base. The existing depth of base was not nearly enough to carry the anticipated traffic even if the new pavement was placed on top. After a thorough study it was found that by placing a 10-inch layer of subbase material,

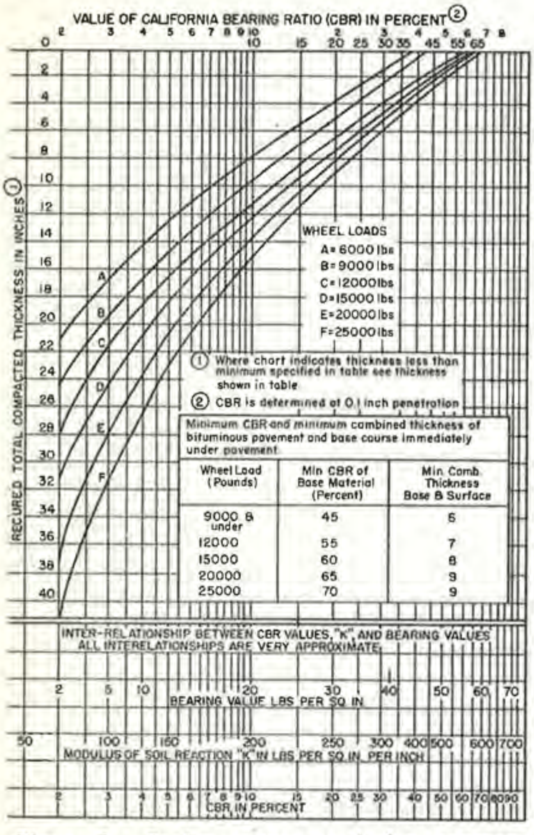


Figure 1. Design curves used for pavement depths required for various wheel loadings.

which was available on the project, on top of the existing base and then placing the new pavement on top of this layer, sufficient depth would be obtained. This subbase material was required to have specified C. B. R. value when tested in accordance with Virginia's method. In doing this the proposed grade was raised an extra 10 inches, which was highly beneficial through the section. Further studies indicated that, by utilizing the existing base material (which was a local sand-gravel mixture) as subbase for the new pavement, the design depth could be satisfied for the other sections where the soils were found with the better C. B. R. values.

The original idea for the design of the pavement was to use a standard depth throughout the entire project. After the C. B. R. studies were made this was all changed. The section of the project with low C. B. R. values was designed to use 20 inches of subbase with an 8-inch asphaltic concrete pavement. In the sections with the better C. B. R. values, the asphaltic pavement was the same; however, the subbase depth was cut to 8 inches. During the construction of the project, close control was maintained to check every change in the material to see that the select subbase material met all requirements and that proper drainage was obtained. Close checks were also made to see that all the materials were well compacted to specified densities prior to the construction of the pavement. The pavement consisted of three types of mixes as follows: $5\frac{1}{2}$ inches of base using the natural pit material with 4 percent asphalt, $1\frac{1}{2}$ inches of binder course with controlled gradation requirements and with 5. 5 percent asphalt and a 1-inch top course of sand asphalt with 7 percent asphalt. The overall depth of pavement was 8 inches. These mixes were also closely checked to insure that the proper stability was being obtained.

It has been agreed that if the original design had been followed the section with the lower subgrade values would not have been strong enough and wholesale failures would have resulted. This project is approximately 3 years old and to date not one failure has occurred. The road is carrying an exceptional amount of heavy truck traffic along with a high count of basic vehicles. Even though the subbase was increased in the poorer sections the overall cost was materially reduced over that which was originally estimated. Certainly the maintenance has been reduced.

After the design of several projects, the question arose as to how much the cost of construction would be increased if this method of design was used. In many instances the initial cost seemed to be greater, but on further investigation it was found that by utilizing the better materials to cap the inferior ones a considerable saving could be made. In following this procedure, the depth of high-type pavement required was materially reduced. This reduction usually more than offset the cost of selective grading. Naturally this is not always the case, especially in areas where the poor soils abound, but it must be granted that if the road is improperly designed excessive maintenance costs would be the result. To date on all the projects that have been designed according to Virginia's method the maintenance has been very low.

Since the method has been employed in Virginia over 75 projects have been designed over all types of soils. The total depth of treatment has ranged up to 48 inches. There have been many cases where the inferior soils have been capped with as much as 36 inches of better material. These better soils were found on the project, or were available close to the work from select borrow sources. The pavement depths have ranged from a minimum of 7 inches to a maximum of 16 inches and have been constructed with all types of base materials and surface course mixtures. In studying the costs involved it is believed that there has been no material increase in overall cost. The plans provided are so detailed that the contractors know exactly what is expected and the works progress very smoothly.

This method of design has been used for all types of construction including reconditioning of all pavements, new locations, and ones where both minor and major changes are made in grades and alignment. On many of the reconditioning projects it has been found that many sections of the old road have to be completely reconstructed, while in other sections only a new surface will have to be applied. On new locations, in many cases, it is necessary to place better materials to depths as great as 24 inches before placing the subbase and pavement. In other instances, where certain grades have to be maintained, it has been necessary to excavate to depths as great as 48 inches and backfill with select material of a known C. B. R. value before constructing the pavement. The idea behind the design method is to provide a uniform subgrade on which to place a uniform depth of pavement that will support anticipated loads with a minimum amount of deflection in the pavement.

Projects that have been designed and constructed to date are very young; however, the results indicate that they are doing the job in a most satisfactory manner. So much so that recent instructions have been to design all projects on Class I and II roads by this method with the hopes that just as soon as personnel and equipment are available that all projects can be designed accordingly. It is realized that this method is not the ultimate but until a more rational one is developed it will be continued in Virginia, since it has proven that it is much better than using the old method based on experience.

Flexible-Pavement Design Correlated with Road Performance

DELBERT L. LACEY, Field Soils Engineer,
Kansas Highway Commission

● THE Kansas method of flexible-pavement design is based upon the results of triaxial tests conducted on each of the components of the road structure shown in Figure 1. By this method tests are conducted on undisturbed soil samples cut from the subgrade and on compacted specimens prepared from materials that will be used for surface, base, and subbase construction. The resulting test values, which are expressed as moduli of deformation in pounds per square inch, are then inserted in a rational formula and the thickness of pavement required to support expected traffic loads is computed.

This procedure parallels the method used to design other engineering structures. The strength value of each of the individual components of the road structure is expressed in pounds per square inch which is comparable to the tensile and compressive strength of concrete, steel, and wood. The major forces that act on the pavement are the traffic wheel loads, which may be predicted with reasonable accuracy and are synonymous with the axle loads acting on a bridge or the floor load within a building.

The formula to which the triaxial-test data is applied is a modification of one developed by Palmer and Barber (1):

$$T \left[= \sqrt{\left[\frac{3P\,mn}{2\pi C S}\right]^2 - a^2} \right] \left[\sqrt[3]{\frac{C}{Cp}} \right]$$

Where:

T = thickness required

Cp = modulus of deformation of pavement or surface course

C = modulus of deformation of subgrade or subbase

P = base wheel load

m = traffic coefficient based on volume of traffic

n = saturation coefficient based on rainfall

a = radius of area of tire contact corresponding to Pm

S = permitted deflection of surface

The modifications of the original Palmer and Barber formula are the addition of the traffic coefficient m, based on the expected volume of traffic, and the saturation coefficient n, based on the amount of annual rainfall in the vicinity of the project under consideration. While these coefficients are empirical, they are based on experience with flexible pavements constructed in Kansas between 1938 and 1942. The traffic coefficient m is based on a state law which limits axle loads to 18,000 lb. and on traffic studies which show that the proportion of commercial vehicles that carry heavy axle loads is fairly constant over most of the state. The range of traffic coefficients used to design the pavements included in this study is shown in Table 1.

TABLE 1

Traffic Coefficient	Wheel Load	Total Traffic
m	Pm lb.	Vehicles per day
1	9,000	1,500 - Up
$\frac{5}{6}$	7,500	900 - 1,500
$\frac{2}{3}$	6,000	300 - 900
$\frac{1}{2}$	4,500	50 - 300

The average annual rainfall in Kansas varies from 16 inches in western areas to 42 inches in eastern sections. Since there is less probability of complete subgrade saturation in the more-arid sections and since service records of flexible pavements indicate better performance in drier areas, the saturation coefficients shown in Table 2 were adopted for the preceding formula.

Since the Kansas method of flexible pavement design has been described in other publications (2, 3, 4) and since this report correlates the method with road performance, details covering the methods of sampling, sample preparation, and testing and design procedures will not be repeated here.

TABLE 2

Saturation Coefficient	Average Annual Rainfall
n	in.
1.0	35.0 -- 45.0
0.9	30.0 -- 34.9
0.8	25.0 -- 29.9
0.7	20.0 -- 24.9
0.6	15.0 -- 19.9

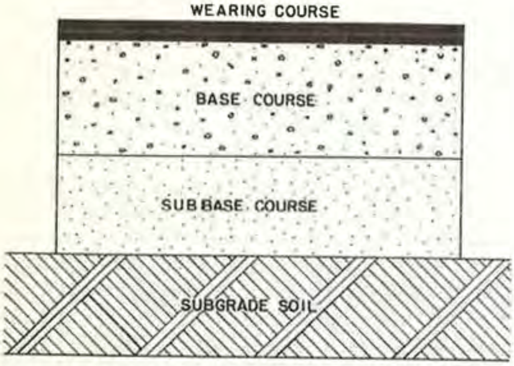


Figure 1. Relative position of pavement components.

This correlation study was conducted on 200 construction projects that totaled 1,243 miles of flexible pavements located in all parts of the state. Of this total, 720 miles of the pavement were designed, by the triaxial methods described above, while most of the remaining 523 miles was constructed between 1938 and 1943, prior to development of the present design method. As might be expected, the subgrade soils beneath these pavements varied from nonplastic dune sands to highly plastic clays in areas where the annual rainfall varied from 16 to 42 inches per year. These pavements have carried traffic volumes that ranged from 180 to 2,800 vehicles per day, of which from 18 to 30 percent were of the commercial variety. A total of nine major base types were represented. These ranged from a layer of selected subgrade soil placed on existing roadbed soils to well-graded sand-gravels stabilized by the addition of asphalt or portland cement.

First, a condition survey was conducted in the early spring, when pavement failures were most pronounced and before major maintenance operations were begun. Thus, a comparison could be made between the surface conditions of the pavements that were designed by the triaxial compression method and those that were not so designed. Furthermore, the empirical terms of the present design formula, the traffic coefficient m and the saturation coefficient n, could be checked against actual road performance.

Figure 2. Class "A" failure.

The condition survey was conducted by two parties of two men each, a driver and a recorder. During the survey each construction project was covered as a separate unit; in order to have a numerical measure of its condition, the total area of base failure as indicated by breaks in the wearing surface was determined and recorded. The dimensions of small failures could be estimated quite closely after some practice. The length of the longer failed areas was measured by car speedometer,

Figure 3. Class "B" failure.

and their widths were either estimated or measured.

Figure 4. Class "C" failure.

Each failed area was classified according to its severity. Where the surface was merely cracked, with no appreciable permanent deformation, as in Figure 2, it was listed as Class A failure. If the cracking was accompanied by a depression of approximately ½ inch, which normally might produce a slight vehicle driver reaction, it was classified as a Type B failure (Fig. 3). When a failure had progressed to the condition shown in Figure 4, where actual rutting had developed with resultant upheaval of the surface beside the normal wheel lane, it was placed in the Type C group. This type of failure was severe enough to cause considerable driver reaction in controlling the vehicle or in trying to dodge the depression.

The total area of each class of failure on each project was computed after the survey was completed. Then the total area of Class A failures was multiplied by 1, the area of Class B failures by 3 and the area of Class C failures by 10, as it was believed that a surface containing numerous Type A failures was less objectionable to the traveling public than one that contained a smaller number of Class C failures. The summation of the equivalent failed areas as determined by the above method was applied in the following formula to compute a survey condition rating for each individual construction project:

$$Rc = \frac{Ap - Af}{Ap}$$

Where:

Rc = survey condition rating
Ap = total surface area of project
Af = equivalent surface area of failures on project

Since under average conditions a maintenance seal is applied to each flexible pavement in Kansas about once each 3 years and since all surface depressions are filled with premixed bituminous patching material before sealing starts, a newly sealed project would have few, if any, surface failures, and there would be little evidence of previous failures.

Therefore, the survey condition rating Rc, as determined by the preceding formula, was introduced into the following formula to compensate for the time that had elapsed since the surface was last sealed:

$$Rs = 1.000 - (1.000 - Rc)\frac{3}{x}$$

Where:

Rs = Maintenance seal rating
Rc = Survey condition rating
x = Number of years since last seal

Thus, if 3 years had elapsed since the last seal, the seal rating (Rs) and the condition rating (Rc) would be equal. Less than 3 years' elapsed time would decrease the seal rating, while more than three years' time would increase the seal rating for the project.

Since failures seldom develop during the first year or two of a pavement's life, it seemed reasonable that credit should be given to older pavements for the service that they have rendered, and since most flexible pavements were planned to have a life expectancy of approximately 10 years, this factor was used to compute a final overall rating for each project by the following formula:

$$Ra = 1.000 - (1.000 = Rs)\frac{10}{Y}$$

Where:

Ra = age rating
Rs = seal rating
Y = number of years since the project was constructed

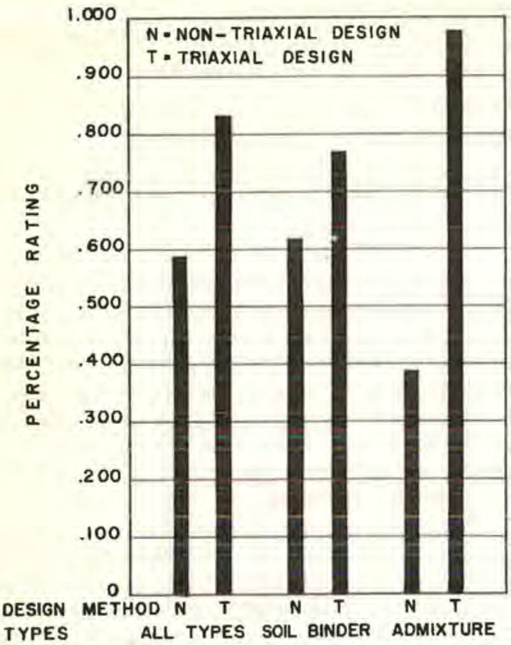


Figure 5. Comparative ratings of base courses.

For pavements that were more than 10 years old, the final rating was increased, while the rating of those that were less than 10 years of age was decreased.

Among the projects covered by this correlation study, a total of 80 projects were not designed by the triaxial method described earlier in this paper. The highest rating found on these was 0.996 whereas a total of seven projects had ratings of 0 or less.

Of the 120 designed by the triaxial method, a total of 29 had ratings of 1, whereas only three had a rating of 0 or less.

In order to summarize the accumulated data, the weighted average rating of each project was computed in accordance with its length. That is, the final rating was multiplied by the project length in miles, then for any combination of projects whose average rating was desired, the individual weighted mile ratings were added and the total sum was divided by the total mileage of the combined projects. This operation was performed to prevent a short project having a very poor or a very good rating from having as much influence on the final overall rating of the group as a very long project of like condition.

The weighted average ratings of all base courses designed by the triaxial method and those not designed by the triaxial method are shown in Figure 5. The triaxially designed projects had an average rating value of over 0.800 while those that were not designed by the triaxial method rated less than 0.600, on the average.

Also shown in Figure 5 are the comparative ratings of two general types of base courses. The two center bars of the graph show the comparative ratings of triaxially and nontriaxially designed bases composed of aggregate bound with soil, while the bars on the right show the average ratings of asphalt and portland-cement base courses.

This figure shows that in each case the pavements that were designed by the triaxial method were in better condition with a resultant higher rating than those not designed by that method. The low rating of the nontriaxial admixture group may be explained by the fact that only a few projects were available for study, and they were the first of each type to be constructed within the state. The type of work was new for both the engineers and the contractors; so with inexperienced inspectors and operators, the quality of construction was somewhat below that which was achieved on later admixture-type bases.

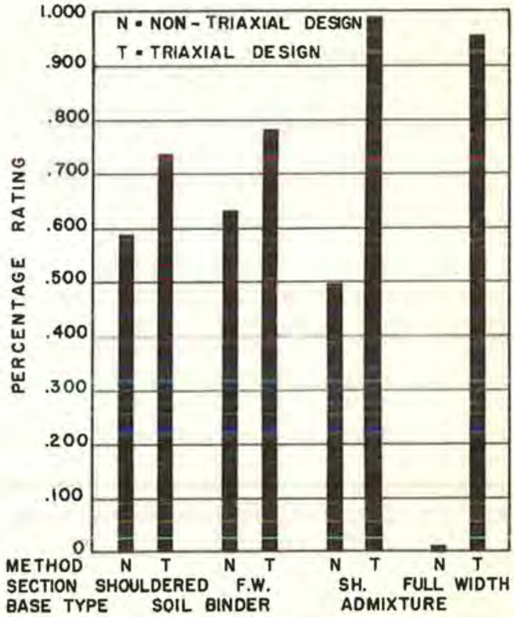
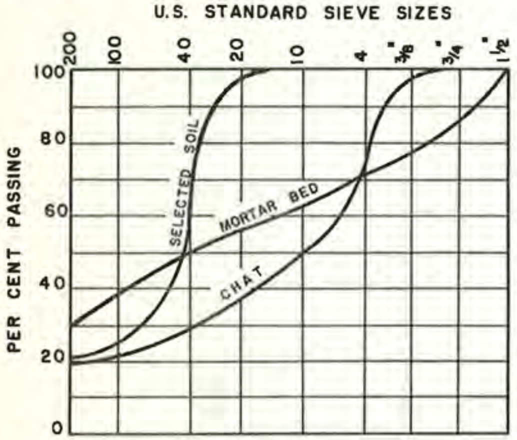


Figure 6. Comparison of shouldered and full width base sections.

Two general pavement sections have been constructed during the past 15 years. Some of the pavements extend entirely across the roadway to permit drainage at their edges, while earth shoulders have been constructed against the outer edges of other pavements to provide parking space for vehicles that are forced to make emergency stops. The comparative average ratings of these two sections are shown in Figure 6.

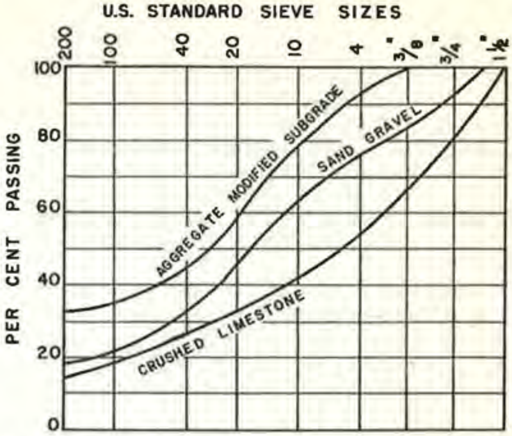
Considering the soil binder base type first, Figure 6 shows that both the full-width and shouldered pavements designed by the triaxial method rate higher than those not so designed. It also shows that the ratings of both the triaxial and non-triaxial full-width pavements are only slightly higher than the ratings of the shouldered pavements. This was rather surprising, as it had been believed that in general the performance of the full-width pavements had been considerably better than those constructed with earth shoulders. This comparatively small difference is, perhaps, due to the fact that a preponderance of the soil binder pavements were constructed in areas of lower rainfall and less-plastic soils where edge drainage is less critical.

It is interesting to observe that the rating difference that exists between the nontriaxially designed full-width and the nontriaxially designed shouldered pavements is almost equal to the difference that exists between the full-width and shouldered pavements that were designed by the triaxial method. In other words, the



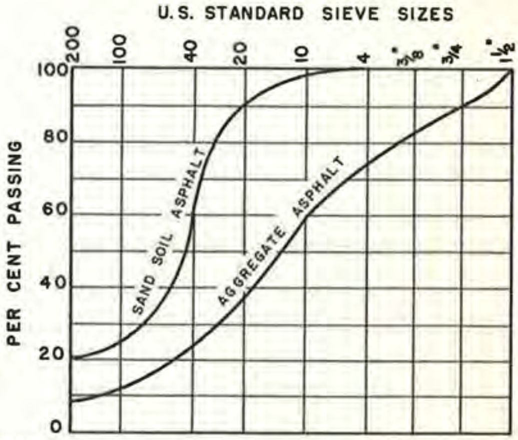
BASE TYPE	LIQUID LIMIT	PLASTIC INDEX	UNIT WT. P. C. F.	RATING NON-TX.	TX.
SELECT SOIL	20	3	123	0.605	0.497
CHAT	21	5	124	.939	.996
MORTAR BED	34	17	112	.819	.704

Figure 7. Typical gradations of soil binder bases.



BASE TYPE	LIQUID LIMIT	PLASTIC INDEX	UNIT WT. P. C. F.	RATING NON-TX.	TX.
AGG. MOD. SUBG.	31	18	125	0.525	0.777
SAND GRAVEL	19	4	132	.693	.743
CRUSHED LIMESTONE	19	5	136	.519	.804

Figure 8. Typical gradations of soil binder bases.

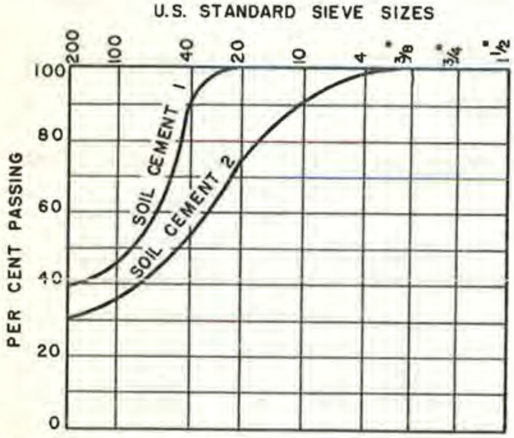


BASE TYPE	LIQUID LIMIT	PLASTIC INDEX	UNIT WT. P. C. F.	RATING NON-TX.	TX.
SAND SOIL ASPHALT	18	4	125	0.312	0.981
AGGREGATE ASPHALT	NON PLASTIC		129	—	.965

Figure 9. Typical gradations of asphalt binder bases.

overall effect of earth shoulders on the performance of soil binder type pavements is about the same for both the triaxially and the nontriaxially designed base courses. This indicates that the procedures originally used in developing the Kansas method of flexible-pavement design were quite effective in correlating the triaxial method with pavement performance.

Figure 6 also shows that the average ratings of the triaxially designed admixture bases were considerably higher for both full-width and shouldered sections than for the soil binder types, with the shouldered sections rating slightly higher than



GRADING CURVE	LIQUID LIMIT	PLASTIC INDEX	UNIT WT. P. C. F.	RATING NON-TX.	RATING TX.
1	16	2	121	0.959	0.998
2	23	12	125		

Figure 10. Typical gradations of soil cement bases.

those constructed full width. This would indicate to a small degree at least, that shouldering has a less-harmful effect on the performance of admixture base courses than on the soil-binder types. The low nontriaxial full-width-base rating is based on only one project, which had an initial survey condition rating of 0. 863. Since the project had been sealed within the past year and since it was only 2 years old, its final rating was low.

The absence of deposits of coarsely graded aggregates in many parts of Kansas has required the use of finely graded sand-gravels, dune sands, and disintegrated sandstones as the primary materials for base construction in some areas in order to hold pavement costs to a minimum. This has resulted in a total of nine general base types within the soil binder and admixture groups, so an opportunity was available to correlate the triaxial design of each type with its performance under traffic.

The textural grading, plasticity, and density characteristics of these various types are shown in Figures 7, 8, 9, and 10. The aggregate-modified base courses were constructed by adding aggregate to the soils that existed in the upper part of the subgrade. Material from local deposits

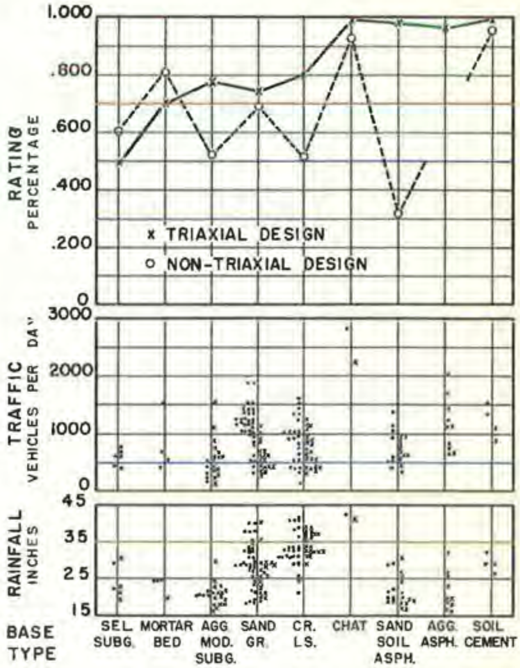


Figure 11. Individual base types, traffic and rainfall.

was hauled on to the project for construction of the other types of base courses.

The comparative average ratings of each of these individual base types and the traffic and rainfall to which each is subjected are shown in Figure 11. This panel shows that, with the exception of the selected subgrade and mortar bed types, the ratings of all the triaxially designed bases were higher than those not designed by the triaxial method.

The lowest-quality base type, that is, the selected subgrade soil, was placed at the left side of Figure 11, and the progressively higher quality base course types were placed to the right. It is evident, then, that the triaxially designed bases constructed of higher quality materials, especially those that were waterproofed with an admixture, have performed better and have developed fewer failures than the base types of lower quality.

More variation occurred in the average surface condition of the various non-triaxially designed base types. The crushed limestone and the sand-soil asphalt bases, which are of higher quality, were lower in rating than the lower-quality base types, such as the selected subgrade, sand-gravel and mortar bed. This indicated that when thickness values were selected for the nontriaxial base courses, a proper balance was not achieved between subgrade stability, base course stability, and traffic loads. The stability of the higher-quality base materials was apparently overestimated.

The formula used in the Kansas method of flexible pavement design has four variables that affect the final thickness of the pavement. These are the traffic coefficient m, the saturation coefficient n, the modulus of deformation of the subgrade C, and the modulus of deformation of the base course Cp.

The correlation of road performance with the modulus of deformation of the subgrade C was not included in this study for the following reasons:

First, to make an accurate evaluation of this variable it would have been necessary to determine the existing subgrade modulus beneath the failed and unfailed areas on each construction project and relate them to the surface condition. Time, personnel, and laboratory facilities were too limited to attempt the collection and testing of the large number of samples that this procedure would have made necessary.

Second, an investigation of this nature was made during the early spring of 1949. During that investigation disturbed base course samples and undisturbed subgrade samples were obtained from seventeen construction projects totaling 248 miles in length. Samples were secured from 17 locations where the surface and base were in excellent condition and from 71 locations where failures were beginning to develop. The results of this study, which have been published elsewhere (4), showed excellent correlation between the subgrade modulus, as determined by triaxial tests conducted on the subgrade samples, and the condition of the base and surface. The subgrade modulus of the majority of the samples obtained from failed areas was, according to the Kansas method, too low to support the overlying base course, while the modulus of the subgrade beneath most of the unfailed sections was equal to or higher than the design method indicated to be necessary to support the overlying base course. Consequently, additional data pertaining to the subgrade modulus was not considered essential to this correlation study.

Third, during design and construction, compensation was made for variation in the modulus of deformation of the subgrade by making corresponding changes in pavement thicknesses. Where the subgrade modulus of deformation was low through a particular section, the base thickness was increased; where it was higher due to an increase in soil stability, the base thickness was correspondingly lowered.

It should be emphasized, however, that it is not practical to specify base thickness changes to compensate for all extremely small localized areas of weak or improperly compacted subgrade soils during flexible-pavement-design procedure. Neither is it practical to construct the entire pavement to sufficient thickness to carry traffic loads over these small localized areas of weak soils. In Kansas it has been the practice to ignore these small areas and to specify a uniform base thickness over sections having considerable length to avoid frequent material quantity changes during construction. Then the Construction Department provides additional base thickness over small areas of weak soil that become apparent as soft spots during material hauling operations or arranges for the removal of the soil before the base is laid down. Occasionally sections are overlooked during the subgrade survey and do not become apparent during construction but appear after the pavement has been in service. It is probable that some of the surface failures logged during this survey were small areas underlain by weak soils that were not found during the subgrade survey or during construction. They are, however, usually so small in proportion to the total area of the project that their effect on the project rating is insignificant.

The average pavement rating for each saturation coefficient for both triaxially and nontriaxially designed pavements are shown in the lower panel of Figure 12. Here again the ratings of pavements designed by the triaxial method are the higher. Since there is no other trend as the saturation coefficients increase from 0.6 to 1.0, it is apparent that the saturation coefficients applied to the climatic conditions that exist in Kansas are valid.

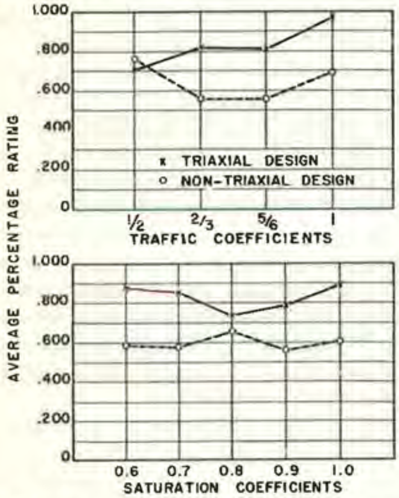


Figure 12. Average rating for each traffic and saturation coefficient.

The upper panel of Figure 12 shows the relationship that exists between pavement rating and the coefficients assigned to the various traffic volumes. The nontriaxial bases show a distinct drop in rating between the one-half and two-thirds coefficients and remain at about 0.600 through the other values. The rating of the triaxially designed base courses increase from slightly over 0.700 to almost 1.000 as the coefficients increase from ½ to 1 for a traffic volume of from 180 to over 2,000 vehicles per day. If this rating line had extended across the chart in a more-nearly horizontal direction it would have indicated closer correlation between traffic volume and pavement performance. The trend shown here does have a definite advantage in that pavements carrying a lower volume of traffic may be constructed at lower cost but with a greater risk of failure, and should failures develop, fewer people will be inconvenienced and repairs will be less expensive. By contrast, the higher-quality, more-expensive admixture types of pavements are specified for more heavily traveled roads; if they were underdesigned to the point of possible failure, more people would be effected and replacement or repairs would be more difficult and more costly.

The relationship that exists between the average modulus of deformation of the base courses included in this study and the base course rating is shown in Figure 13. A sufficient number of stability tests have

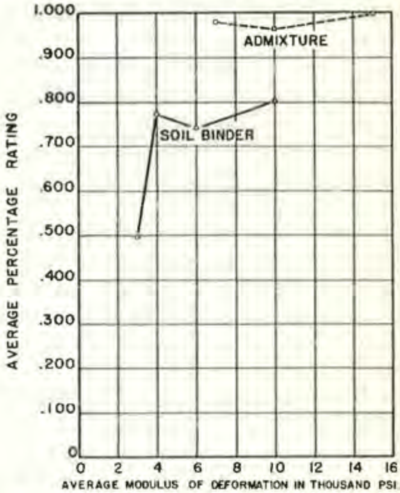


Figure 13. Comparative ratings of modulus of deformation for each base type.

been conducted to establish an average modulus of deformation for each type of base, and these values were used in preparing this chart. This figure shows that the rating of the lower quality pavements (those with lower moduli) is lower than the ones with higher moduli. It is also interesting to note that, where the two types have the same moduli of deformation, the admixture type has the higher rating, indicating the value of the admixture as a waterproofing agent. This would also indicate that if these two general types of pavement are to render the same degree of service under traffic, some revision in coefficients may be in order. Detailed studies of a large number of individual projects must be conducted to determine the nature of these changes. This study will need to include the degree of saturation of the several different base types, as well as the degree of saturation of the subgrade beneath each type to determine the relative amount of protection that each base provides for the underlying subgrade. Such a study is tentatively planned to establish closer correlation between performance of base courses and their moduli of deformation.

It should be emphasized that many variables were entirely ignored during this survey. A few of these variables were: (1) weather conditions during construction, (2) quality of material used during construction, (3) relative degree of maintenance that each project has received, (4) effectiveness of the surface and subsurface drainage systems, and (5) weather conditions under which the bases have performed. For example, many weather stations in Kansas recorded considerably more rainfall during the years between 1941 and 1951 than during any other similar period of weather-bureau history. The rainfall in 1951, when many of these bases were in service, ranged from 130 to more than 175 percent of normal; during periods of flood many arterial highways were closed, and traffic was diverted on to routes whose bases had not been designed to carry such large volumes of traffic. None of these variables and their effect on the pavement condition were taken into consideration during this correlation study.

In conclusion, it may be stated that the data collected during this correlation study indicates that: (1) Pavements designed by the Kansas method of flexible-pavement design generally have performed better than those not so designed. (2) The Kansas method of flexible-pavement design has been successful over a wide range of soil, climatic, and traffic conditions. (3) The method properly evaluates the stability of a wide variety of base course materials, a factor that is especially advantageous in areas where local deposits of coarsely graded aggregates commonly used for pavement construction do not exist. (4) Some revision of coefficients or test procedure may be indicated to equalize the design of soil binder and admixture type bases so their performance will be more comparable. (5) By the use of proper coefficients based on correlation with pavements in service, the method may be adapted to fit climatic and traffic conditions that exist in other areas (5).

ACKNOWLEDGMENTS

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General Discussion

W. S. HOUSEL, Professor of Civil Engineering, University of Michigan — In opening the discussion, the writer as chairman has a few general comments. In reviewing these papers, one is impressed that there are several significant trends which are becoming apparent in the development of flexible-pavement design.

In the first place, most of the states now reporting have found that the designs which have been used over the past 5 or 6 years have been proven inadequate. These states are now engaged in revising their designs to provide greater pavement thickness in order to decrease the percentage of failure or increase the useful life of the paving. These early failures appear to have resulted from inadequate thickness which is generally felt to be largely due to an unanticipated increase in the volume of traffic accompanied by increasing wheel loads and the percentage of heavy loads.

This calls attention to the fact that all states reporting have recognized the importance of the volume and character of the traffic but have, in most cases, underestimated its rapid development. It also raises a question as to whether or not the intangible aspects of traffic on the roadway structure have been properly evaluated in principle as well as magnitude. This same question comes up again when studying the various revisions which are now suggested to modify design procedures which have been in use. In some cases, revisions are being suggested in design procedures which involve the fundamental principles on which the method has been predicated in such a way as to seriously compromise those principles. In several cases, the compromise comes about through modification of the basic formula by statistical or numerical coefficients which are not directly related to these principles so both the principles and the modifying conditions lose their identity.

Basically all methods must be built around certain fundamental principles involving the structural mechanics of a flexible surface. These principles find expression in certain physical factors or properties of the surfaces or the materials of which they are built. These properties are subject to quantitative measurement by one or perhaps several test procedures which are relatively precise and capable of duplication in any well-equipped laboratory. Such design factors should be relatively stable and not subject to frequent and broad revision. Otherwise, they lose their value as a basis of reference to which widely varying experience can be compared.

On the other hand, there are several factors or conditions affecting pavement performance which possess certain intangible aspects not subject to precise measurement. These conditions may be equally important insofar as pavement performance is concerned, but they are distinctly different in character and cannot be incorporated in the structural mechanics of the flexible surface. The volume and character of traffic loads and certain environmental factors fall into this classification.

It seems to the writer that the papers of this symposium and the experience of the 6 or 7 years which they represent make this quite clear. In this connection, the design methods used by Wyoming and Kansas represent the most-comprehensive attempt to express in a combined form both the structural mechanics of a flexible pavement and the modifying conditions which affect pavement performance.

In the Wyoming method which is similar to that used in Colorado, the basis of design is the CBR value or rather a modification of the conventional CBR procedure. Eight different design or wheel load curves have been prepared giving total combined thickness of surface for CBR values from 3 to 100. "Selection of the wheel load curve to be used is indicated by the total empirical (numerical) value obtained from evaluating job conditions and traffic within the section." Annual precipitation is represented by numbers from 0 to 10, depth of water table, 0 to 5, frost action, 0 to 8 and other existing conditions, 0 to 6. Traffic is converted to equivalent 5,000-lb. wheel loads by certain numerical factors which, in turn, are translated into a design value number varying from 0 to 24. A composite number obtained by adding together the above numerical components indicates the design curve to be used.

The Kansas method makes use of the triaxial-compression test as the primary basis of design. The pavement-thickness formula used has a rational background but is modified by numerical coefficients for traffic volume and saturation as related to rainfall which converts the formula to an empirical basis. The traffic volume coefficient, m, varies from ½ to 1 and the saturation coefficient, n, varies from 0.6 to 1.0. A somewhat similar technique is employed in rating pavement performance. Class A, B and C failures are recognized grading from those of least severity to the most-severe rutting and displacement. Then the total area of Class A failures are multiplied by one, Class B by three and Class C by ten to obtain an equivalent area of failure. The condition rating is then the difference between the total area and the equivalent area of pavement failure divided by the total area in the project.

While the attempt to establish a numerical rating for a project which includes a systematic evaluation of each and every factor which affects pavement performance is to be commended, the result obtained raises a number of questions. As far as the writer is concerned, the mechanics of the pavement structure appears to have been so submerged by numerical ratings and coefficients that the result is no longer definitive of structural behavior. Insofar as pavement performance is concerned, the complex numbers convey no mental picture of the actual condition of the pavement. In other words, the perspective of the entire problem and the relation between its component parts seems to have been lost.

Turning to the other methods reported, Virginia and Missouri have left the intangible factors of traffic volume and environment in the broad category of engineering judgment. While this is conveniently indefinite, it at least is not confusing. Woodson in Virginia has stayed on the conservative side and utilized selective grading and highly commendable use of the better materials where they would do the most good. This places the level of his design very high and failures are held to a minimum. Thus, the Virginia practice as reported is the one exception where the pavement design reported does not require revision to provide increased load-carrying capacity.

Hicks in North Carolina in employing full scale loading tests presents a method of design which approaches the so-called rational method which seems to be the ultimate objective of flexible-pavement design. Environmental factors and particularly drainage are controlled by regulating the moisture content and placement of subgrade soils in the test section. The fact that traffic loading has been underestimated is the reason that Hicks is now working out a revision of his method to produce a thicker pavement on a certain class of roads. It seems unfortunate that this revision has become involved with pressure-distribution laws which are relatively inflexible relationships and not logically subject to substantial change. Perhaps some other means of adjusting the design may be found.

McDowell in reporting the method of design developed in Texas used a device which suggests one logical solution to the difficulties under discussion. The Texas design, insofar as structural mechanics is concerned, utilizes the triaxial-compression test to evaluate the resistance characteristics of subgrade soils and flexible-base materials. The Texas design is based on the theory that if none of the components is stressed beyond its elastic limit, the road will withstand a practically unlimited number of stress repetitions without failure. From the traffic survey, the average ten heaviest loads per day is determined and this loading used to establish what may be referred to as the basic or 100-percent-design thickness. In the case of Texas, the estimated life of such a pavement is 28 years, but this appears to be a rather hypothetical figure and may involve deterioration other than structural failure due to loading.

For the purposes of this discussion, this design would be considered too heavy for many classes of roads and would probably be limited to the most-important trunk lines, if used at all. McDowell introduces a term called "percentage of design" which simply designates a recommended pavement section selected as a percentage of the basic design. This percentage of design provides complete flexibility in adjusting the pavement section actually constructed in any given case to fit the traffic volume and percentage of heavy loads or other of the intangible factors, without disturbing the basic design. If such a percentage-design method had been in general use in the various states that now find from condition surveys that they have been designing too light, it would now merely be necessary to increase the percentage of the basic design used in any selected project.

The adoption of such a procedure is strongly recommended as a possible solution to some of the most-perplexing problems which the designer of flexible pavements must face. If the volume and character of traffic and the intangible environmental factors can be set apart for such separate treatment, the problem of correlating soil resistance and the structural mechanics of flexible pavements will be greatly simplified. In the work of this committee, it might then be possible to establish test procedures of one of several of the recognized tests so that there would be reasonable duplication of results and reasonable agreement on a basic design for any given soil by any laboratory or designer regardless of where located. Traffic characteristics as affected by locality and other local environment could then be delegated to the designer best acquainted with those special conditions. This designer's adjustment of the basic design on a percentage basis would then be much simplified and readily understood by all who might attempt to review and compare flexible pavement design on a country-wide basis.

JOHN M. GRIFFITH, Engineer of Research, The Asphalt Institute—It seems as though there are about as many procedures for determining thicknesses of flexible pavements as there are of skinning a cat. Several of these procedures have been described here and others are conspicuous by their absence. Perhaps it just depends on the type of car you wish to drive or the road you wish to travel.

All methods are intended to arrive at the same destination: adequate thicknesses of available materials to support anticipated wheel loads and volume of traffic satisfactorily under applicable climatic conditions and under economic conditions of first cost and maintenance. Although "rationality" is claimed by some, all of the existing methods should be considered as empirical in their present state of development. There are a multitude of variable factors entering into this problem which cannot be fully evaluated by existing procedures and so we must take the empirical approach by devising a testing procedure and then correlating it with experience. Woodson has recognized this to be the case insofar as Virginia is concerned. For example, how would we rationally evaluate the loss in bearing capacity resulting from so-called frost action which few even mention in their discussion? Also, most test procedures involve only a single load application or a series of increasing loads, whereas we know that a flexible pavement is subjected to an indeterminate number of repetitive loads. Does a direct relationship exist between test results from single load applications and the repetitive applications of traffic? Perhaps a truly rational approach may prove to be too cumbersome to even find utility in practical application.

This reasoning, however, should not be taken as a defeatist attitude or as an implication that the empirical test cannot be adjusted to provide entirely satisfactory and economical answers. To the contrary, the papers indicate that the several empirical approaches are providing satisfactory answers when satisfactorily adjusted for local conditions of climate and traffic. Perhaps some are too near the borderline and result in areas of unsatisfactory performance, but in time they may be adjusted to provide satisfactory and economical answers. And it would seem to make little difference as to the mode of transportation or the route, if the proper destination is reached.

Davis points out that they use the group-index-classification method, although Missouri's experience with this method has been rather limited to date. It is interesting to note that he did not mention consideration of frost effects although he does mention that frost penetration varies from 5 inches to 20 inches in his state and these depths encompass the range of thicknesses often employed over the native subgrades. He also notes that most of the pavements designed by their methods to date have been over silty soils, usually considered as bad actors, insofar as frost is concerned. Another influencing factor, apparently, not adequately considered is drainage. Good drainage is an important factor in the behavior of any type of pavement.

Both McDowell and Lacey report the successful use of the triaxial test— but by rather widely varying methods. Extensive surveys in both states have indicated a good batting average for this often called "rational" test procedure, despite the fact that different approaches were used both in testing procedure and in application of the data. Apparently both Texas and Kansas have adjusted their respective methods to be applicable to local environmental and traffic conditions. No mention was made by either author on the evaluation of frost effects in their procedures, although perhaps it is taken into account. This factor would seem to be of some importance in Kansas and, also, in certain areas of Texas.

Both Woodson and Olinger seem to have confidence in so-called modified CBR approaches, although their respective modifications are somewhat different. Woodson points out that Virginia's modified CBR approach has been used on over 75 projects, apparently with considerable success as he mentions that maintenance costs have been quite low. He recognizes that the procedure is not the ultimate but is considered to be adequate until a "more rational" method is developed. Olinger points out that Wyoming's modified CBR approach has been used on about 1,450 miles of roads in the past 7 years with greater than 95 percent of satisfactoriness. Also, he appears to adequately recognize the contingent factors wherein unsatisfactory performance was experienced, and it is to be expected that these factors will be given due recognition in the future.

Hicks has taken the commendable approach of recognizing the limitations of the usual, small, cylindrical test specimens and he makes tests in North Carolina on 42-inch-square specimens, 30 inches deep, and compacted to simulate expected conditions in the roadbed. This should do much toward eliminating the confining effects of molds on test results and provide answers more nearly approaching those which would be achieved in field testing. In his analyses, Hicks has used a pressure-distribution theory which is an oversimplification and is incorrect. In discussing this matter with Hicks, however, he has indicated that he recognizes this deficiency and plans to make appropriate adjustments.

But perhaps there is a single best method which will become recognized by the highway engineer. The scientific approach to road design and even the science of soil mechanics itself on which scientific road design is based are relative infants in the scientific field. Both are growing fast, however, and are rapidly developing into respectable members of the scientific family. Papers such as have been presented here are essential to this growth and are of immeasurable assistance in leading us toward the most-appropriate solution to this important problem.

CHESTER McDOWELL, Closure — Apparently Griffith has studied the various flexible-pavement reports rather thoroughly; however, there are two points which should be pursued further. One of these is that the methods failed to discuss the effects of "frost" upon pavement design. The author agrees that he should have included at least a limited statement on this subject. Occasionally, we have "freeze damage" which disturbs extensively the upper portion of borderline-quality bases under thin surfacings. After thawing, the pavement is cracked badly and most support from the upper portion of base is lost. It has been interesting to note that to date we have not found a single case where this trouble has occurred if such thin surfacings are placed on Class 1 base materials. When so-called frost damage penetrates deeply into subgrade, the expansive pressures generated are too great for any reasonable thickness of pavement to have weight or strength enough to resist them. Therefore, the problem is one of excavating and wasting of all soils susceptible to such damage down to the frost penetration line or wrapping them up with impervious membranes. The only alternative, but not a solution to the problem, is to use a type of pavement which can be reworked easily and economically after damage occurs.

Another important point mentioned by Griffith pertains to the effects of repetitive wheel or axle load applications. The Texas design method is based upon the theory that elastic bodies recover from an enormous number of deflections caused by loads, provided each layer in the pavement system is not overstressed. The pattern of normal highway traffic is such that it would be difficult to select the heaviest wheel load to be applied repetitively to design unless some average of the heaviest loads is made. For this purpose we use the average of the ten heaviest wheel loads per average day.

This average is a stable figure which planning survey divisions can obtain and is one which enables us to design for the heaviest loads being applied repetitively. Even though we design for an elastic body supporting repetitive loads, Figure II of the report would indicate that there is a limit to the life of such pavements which is about 28 years. It does not seem illogical to expect a pavement to eventually suffer from fatigue even though it is supported by an elastic medium. It is also interesting to note that pavements having lower percentages of design also have much worthwhile service life in them, even though they are not designed for elastic conditions. These types of thinner pavements suffer slight amounts of permanent deflections from repetitive loadings before failures are noticeable. It is becoming more evident that designers should anticipate the life expectancy required of a road before thicknesses of pavements are determined. The author does not believe that the use of a wheel-load-design criterion equal to the average of the ten heaviest per average day is applicable to all patterns of traffic; therefore, the following expression taken from Figure II becomes useful:

$$\text{Number of years life} = 0.96(10)^{.01465\ (\%\ \text{design})}$$

(Substituting 10 heaviest wheel loads per day)

The number of load applications =

$$10 \times 365 \times 0.96(10)^{.01465\ (\%\ \text{design})}$$

Using our 100-percent design criteria for 28 years' service, which is based on normal traffic containing approximately 25 percent of commercial traffic, the number of applications =

$$3504 \times 10^{1.465} = 3504 \times 29.18 = 102,247.$$

If the amount of commercial traffic doubles, it is possible to design for 20 or any other number of load applications per day desired. The following tabulation shows some estimates which can be calculated from the above formula:

Commercial Traffic				
Proportion of Commercial Traffic	Percent based on Total Including Fixed No. of Cars	Number of Applications of Ten Heaviest Wheel Loads in 28 Years	Percent Design Indicated	No. Years Life Expected If 100% Design Is Used
Normal	25	102,274	100	28
2X Normal	40	204,494	120	14
4X Normal	49	408,988	141	7

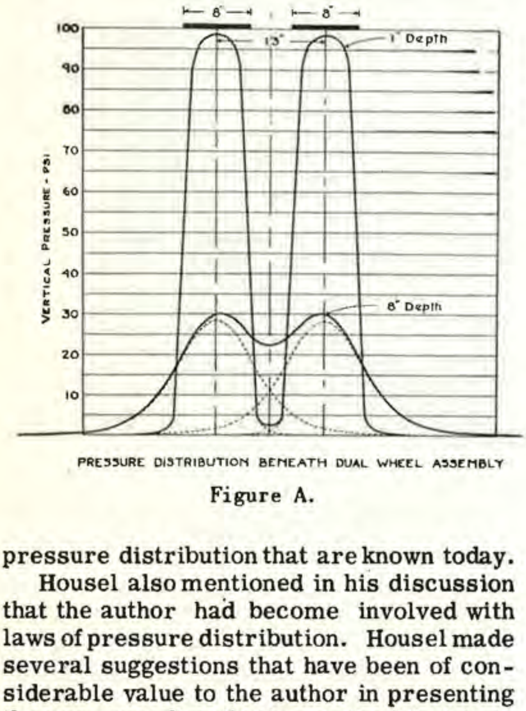
We are indebted to Griffith for calling these points to our attention.

Herner's discussion is pertinent and many of our findings agree with his. It is agreed that some pavements stand up for years under comparatively light traffic, then fail within a short period of time when loads are increased. In analyzing problems of this type it is desirable to calculate a new percentage of design based on the increased loads and consider only the time the heavier loads have used the pavement as the life of the pavement. It is also suggested by Herner that only a narrow weight range of loads be averaged. It is for this reason that we average only the ten heaviest per average day. Herner is correct, and as suggested in the above method, it is possible to break the study of pavement life into separate intervals. This has been done a few times and the results produced the same trend as shown in Figure II. Our difficulty with using this approach lies in obtaining accurate traffic data.

Among other things, Herner mentions the effects of height-diameter ratio upon results of triaxial tests and he also mentions that they use 10-inch diameter by 20-inch and other heights of specimens. The data shown by him on this subject seem in reasonable agreement with the limited data we have accumulated in testing 6-inch-diameter specimens having various heights. Although the data available to the author are not conclusive, the following is believed to be true of granular materials: (1) Due to frictional interlocking, specimens consisting of such materials when molded in 6-inch-diameter molds cannot be compacted as thoroughly as they can when formed in ten-inch diameter molds. (2) For strengths to be comparable it is likely that 10-inch-diameter specimens will require greater height-to-diameter ratio than 6-inch-diameter specimens.

The use of such large specimens as have been suggested would require nearly seven times as much material as we presently use, and each specimen, plus accessories, would be too heavy to handle manually. Although large specimens are desirable in many respects, it would be difficult to provide our 25 district laboratories with the equipment necessary for running such triaxial tests. Herner and our laboratory have agreed to pursue this problem further by exchange of samples and information in the future. The engineering profession is fortunate to obtain valuable comments and splendid data from one as well versed in this subject.

L. D. HICKS, Closure—In his discussion of the papers presented in this symposium, Griffith states that the author in his analysis "has used a pressure-distribution theory which is an oversimplification and is incorrect." In a private discussion of this matter, before Griffith presented his written discussion, the author admitted that the simplified pressure distribution used did not follow the rigorous laws of pressure-distribution theory and stated that he planned to use another method of pressure distribution that would be more acceptable because it followed the recognized laws of



PRESSURE DISTRIBUTION BENEATH DUAL WHEEL ASSEMBLY

Figure A.

pressure distribution that are known today.

Housel also mentioned in his discussion that the author had become involved with laws of pressure distribution. Housel made several suggestions that have been of considerable value to the author in presenting the pressure distribution method that is to follow.

Experiments in pressure distribution have shown that areas with uniform pressure produce helmet-shaped pressure profiles on horizontal planes beneath the loaded area, the greatest pressure occurring beneath the center. Computations using formulas derived from the theory of elasticity verify this. Among the methods for computing pressures beneath uniformly loaded circular areas is one developed by N. M. Newmark of the University of Illinois. He developed a chart which can be

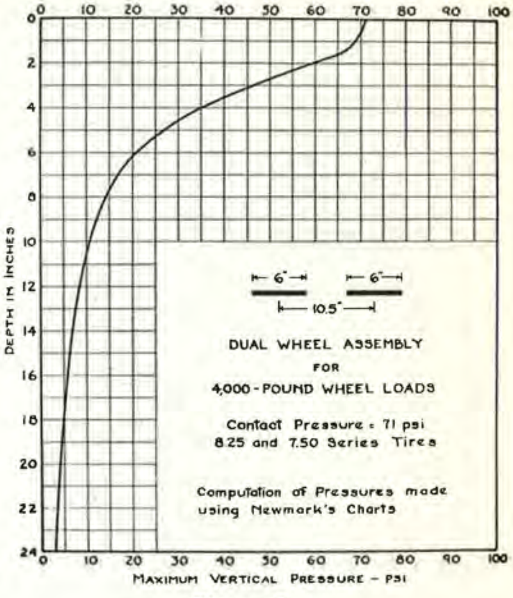


Figure B.

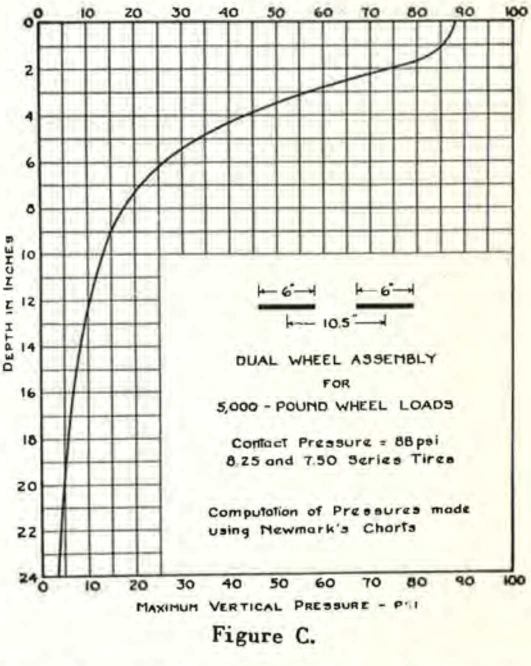


Figure C.

used to compute the magnitude of pressure on any point of any plane beneath the area. The Corps of Engineers of the U. S. Army in Technical Memorandum No. 3-323 developed curves from Newmark's chart that are more convenient to use in certain cases. The author has made use of these curves in his computations of maximum vertical pressure beneath dual wheel assemblies commonly used on trucks carrying highway loads.

Figure A shows the pressure distribution beneath a dual-wheel assembly consisting of two 11.00 series tires carrying 5,000 lb. each (pressure = 100 psi.). Two pressure profiles are shown at depths of 1 and 8 inches. The dotted lines denote the pressures caused by each tire and the solid lines the total pressures caused by the entire assembly.

Figures B through I are curves showing maximum vertical pressures beneath dual-

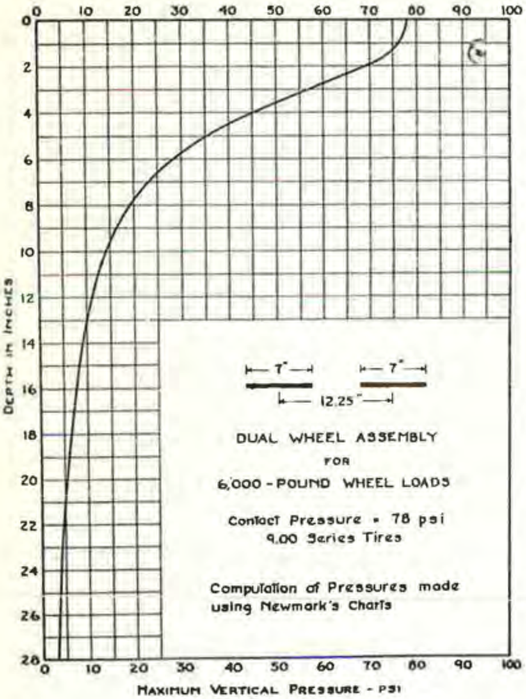


Figure D.

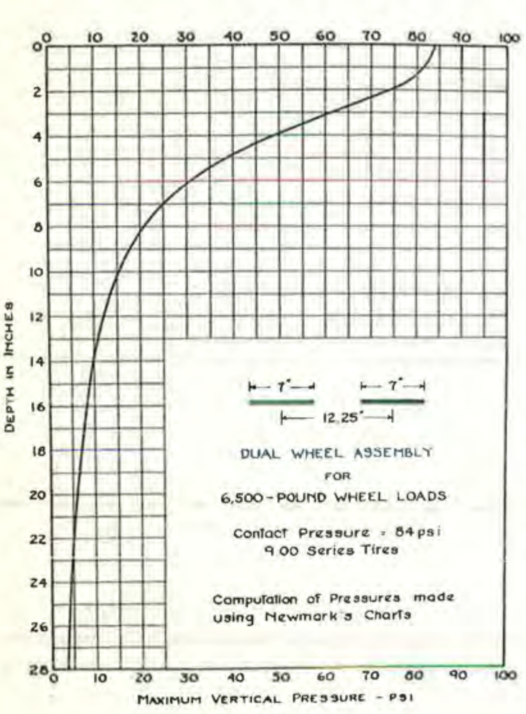


Figure E.

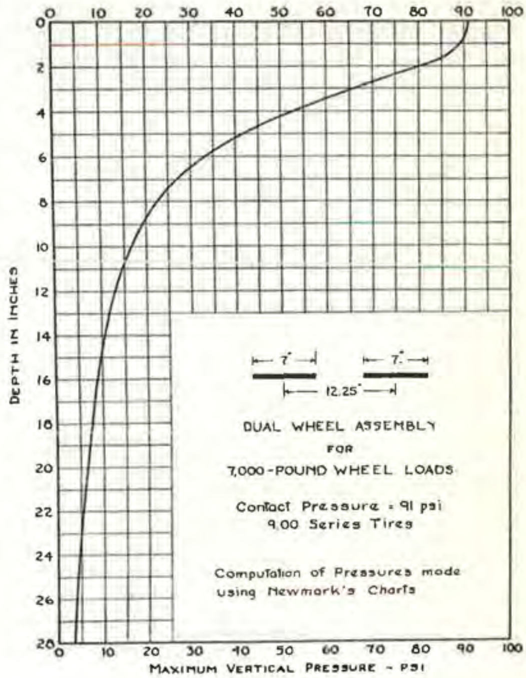


Figure F.

wheel assemblies that carry wheel loads of 4,000, 5,000, 6,000, 6,500, 7,000, 8,000, 9,000, and 10,000 lb. The tire size and wheel assembly selected for computing the maximum vertical pressure for each of the wheel loads are those commonly used for these loads. The contact areas are assumed to be circular and having diameters equal to the width of tread. The contact pressure is calculated by dividing the tire load by the area of contact. Computations of pressure as mentioned above, were made using the curves developed from Newmark's charts by the Corps of Engineers, U. S. Army and appearing in Technical Memorandum No. 3-323.

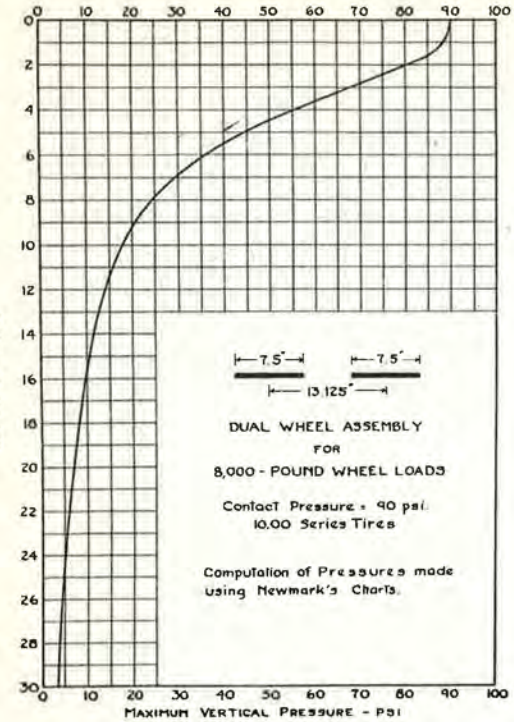


Figure G.

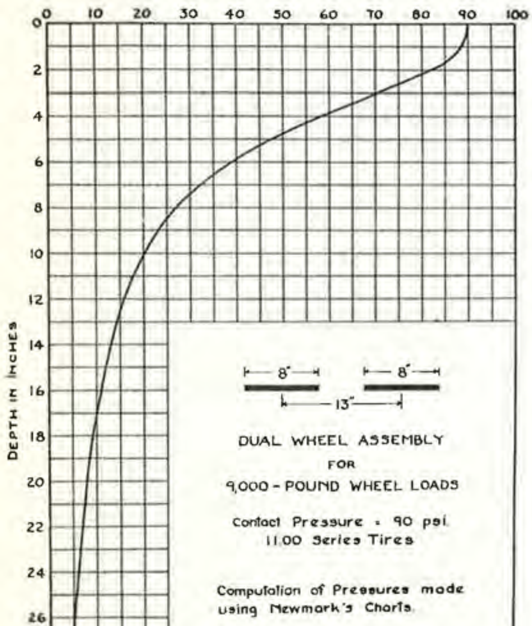


Figure H.

The use of the curves shown in Figures B through I for determining the thickness of a flexible pavement is quite simple. The total thickness of pavement required to carry a certain wheel load is equal to the depth below the surface of contact where the pressure is just less than the bearing capacity of the subgrade soil. For example, let it be required to determine the thickness of pavement necessary to carry 10,000-lb. wheel loads when the bearing capacity of the subgrade is 20 psi. Referring to Figure I the curve showing the

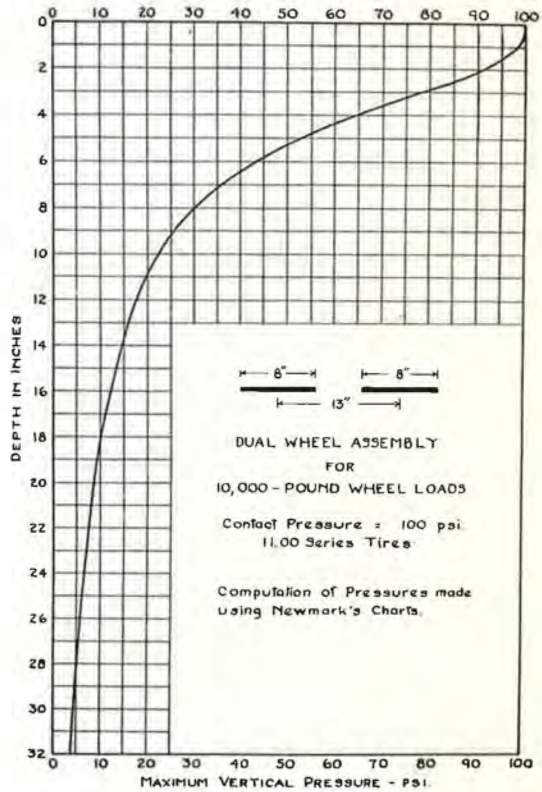


Figure I.

maximum vertical pressure beneath a dual wheel assembly loaded to 10,000 lb., we find that at a depth of 11 inches the pressure is 20 psi., so the minimum total thickness of pavement required is 11 inches.

The curves may also be used for determining the pressure anywhere within the pavement structure, which is essential in selecting the type of base course or sub-base course. For instance, if in the above problem a 3-inch asphalt surface course is to be used, the base course should consist of 8 inches of material capable of developing a bearing capacity of at least 78 psi., as the pressure 3 inches below the surface is 78 psi. Similarly, if a cheap material is available locally that is capable of developing a bearing capacity of at least 30 psi., it may be used to construct a 3-inch subbase. The pavement design in this instance can consist of a 3-inch subbase of local material, a 5-inch stone or gravel base, and a 3-inch asphalt wearing surface.

It will be recalled that this same problem was used as an example in the main paper, and by a coincidence the same total thickness of pavement was obtained using the original design method. It will also be recalled that the writer stated that this

TABLE A
(Revision of Table 1)

TENTATIVE SUBGRADE BEARING VALUES

Subgrade Bearing Values	Description of Soil
20 - 30 psi.	Generally sandy soils of the A-2 and A-3 groups. Containing less than 35% material passing a No. 200 sieve. CBR values between 10 and 20. Good drainage. Subgrade should be below the frost line. * Thickness of stratum must not be less than 12 inches.
15 - 20 psi.	Silt-clay soils with some plasticity, of the A-4, A-5, A-6, and A-7 groups. Clay content generally less than 50%. CBR values between 7 and 10. Good drainage. Subgrade should be below the frost line. * Thickness of stratum must not be less than 12 inches.
10 - 15 psi.	Heavy clay soils, clay content generally above 50%. Quite plastic. CBR values between 4 and 7. Good drainage. Subgrade should be below frost line. * Thickness of stratum must not be less than 12 inches.
5 - 10 psi.	Non plastic, silty and micaceous soils of the A-4 and A-5 groups. Also other soils having questionable drainage. CBR values between 2 and 4. Good drainage. Subgrade should be below frost line. * Thickness of stratum must not be less than 12 inches.

*Frost line is the depth below which water in the soil will not freeze.

thickness was found to be inadequate for a particular road whose subgrade had been rated as having a bearing capacity of 20 psi., and used the method of pressure distribution described in the main paper, which produced a thickness of 16 inches, that has proved adequate. These facts lead to one conclusion, that the subgrade rating has been too general, resulting in higher ratings than the actual bearing capacity of some of the soils. After considerable checking and investigation, the writer has found this to be true and as a result has revised the tentative subgrade-bearing values appearing in Table 1. These revised values appear in Table A of this discussion.

In order to more accurately select a bearing capacity rating value for a subgrade for use in designing the thickness of pavement required, CBR tests are now made on subgrade samples at the proper moisture content for the particular soil (see PROCEEDINGS, Highway Research Board, Vol. 28, p. 430, (1948), Table 8). These values are converted into bearing capacity values by the use of the curve shown in Figure J. It is felt that this procedure will enable the writer to be more specific and less general in selecting the bearing capacity values of subgrades on projects he is called upon to

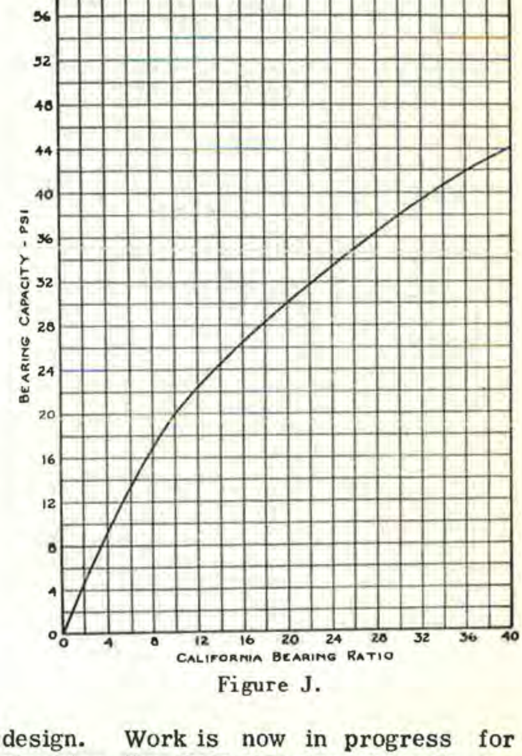


Figure J.

design. Work is now in progress for checking the CBR-Bearing Capacity relationship curve shown in Figure J.

In connection with the above problem an investigation revealed that many of the soils which had been rated as having bearing capacities of 20 psi. were found to have CBR values of about 5 when tested at their proper moisture contents. Referring to the curve in Figure J, it is found that this CBR value indicates a bearing capacity value of 12. Using this value of bearing capacity with the curve shown in Figure I, a thickness of 16 inches is obtained, which thickness has been found adequate.

It might be mentioned that using a bearing capacity of 12 instead of 20 in Vokac's formula for pavement thickness a thickness of 15.3 inches is obtained.

$$h = \frac{b}{2}\sqrt{\frac{P}{P_o} - 1}$$

$$h = \frac{11.3}{2}\sqrt{\frac{100}{12} - 1} = 15.3 \text{ inches}$$

In conclusion, the writer is of the opinion that the proper selection of subgrade bearing capacity is probably the most-important factor in determining adequate pavement thicknesses. Of course, the design load and method of pressure distribution have their importance, but overestimating the strength of a subgrade can cause more failures than using simplified pressure distribution concepts that do not follow exactly the rigorous laws of that science. Although the writer is abandoning the Vokac pressure-distribution concept in favor of the one used in this closure, he is nevertheless planning to do considerable work on the bearing capacity of subgrade soils in order that safe values will be used in his pavement thickness design in the future. The curves shown in Figures B through I required a considerable amount of work in computing the maximum vertical pressures, but this work is compensated for in the simplicity of their use and the satisfaction that they do obey the rigorous and inflexible laws of pressure distribution. The writer wishes to thank Housel for his suggestions and advice in preparing this closure and plans to follow along these lines of flexible pavement design in his future work. More may be said in a future article.

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