MoDOT

TE
5092
.M8A3
no.69-7
c.2

4, 26, 55, 57

MISSOURI COOPERATIVE HIGHWAY RESEARCH PROGRAM
REPORT 69-7

EVALUATION OF LIME AND PHOSPHORIC ACID STABILIZATION IN MISSOURI

MISSOURI STATE HIGHWAY DEPARTMENT
BUREAU OF PUBLIC ROADS

Property of
MoDOT TRANSPORTATION
LIBRARY

EVALUATION OF LIME AND PHOSPHORIC ACID STABILIZATION IN MISSOURI ROUTE 169, GENTRY COUNTY

STUDY 63-1

Prepared by

MISSOURI STATE HIGHWAY DEPARTMENT
Division of Materials and Research

JUNE 1969

in cooperation with

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

The opinions, findings, and conclusions expressed in this publication are not necessarily those of the Bureau of Public Roads.

TABLE OF CONTENTS

	Page
List of tables	I
List of figures	II
Introduction	1
Findings	4
Discussion of data	6
Mathematical model adopted for analysis of Benkelman beam deflections	12
Analysis of variance of L/2UH	24
Equivalent thickness of pavement, H_e	32
Analysis of variance of $L/2UH_e$	43
Computed values of subgrade modulus	50
Equivalent thickness of pavement required to restrict deflections to a tolerable level	55
Performance of test sections	61
References	71

LIST OF TABLES

Table		Page
1	- Some physical characteristics of the roadway	3
2A	- Average values of Benkelman beam deflections for first set of test sections.	8
2B	- Average values of deflections for second set of test sections.	9
3A	- Average ratios of load to deflection, L/2U, for first set of test sections.	17
3B	- Average ratios of load to deflection for second set of test sections.	18
4A	- Average values of pavement unit strength, L/2UH, for first set of test sections.	19
4B	- Average values of pavement unit strength for second set of test sections.	20
5	- Analysis of variance of pavement unit strength.	25
6	- Notation for analysis of variance.	27
7	- Values for the equivalent thickness of pavement, H_e.	34
8	- Values for the subbase equivalent thickness of rolled stone.	39
9A	- Average values of normalized unit strength, $L/2UH_e$, for first set of test sections.	41
9B	- Average values of normalized unit strength for second set of test sections.	42
10	- Analysis of variance of normalized unit strength.	44
11	- Summary of normalized unit strength and summary of traffic variables.	49
12	- Computed values of subgrade modulus.	52
13	- Tentative tolerable deflections.	58
14	- Equivalent thickness required in each wheelpath.	62
15	- Values from condition survey of test sections.	63
16	- Expected service lives of test sections.	66

LIST OF FIGURES

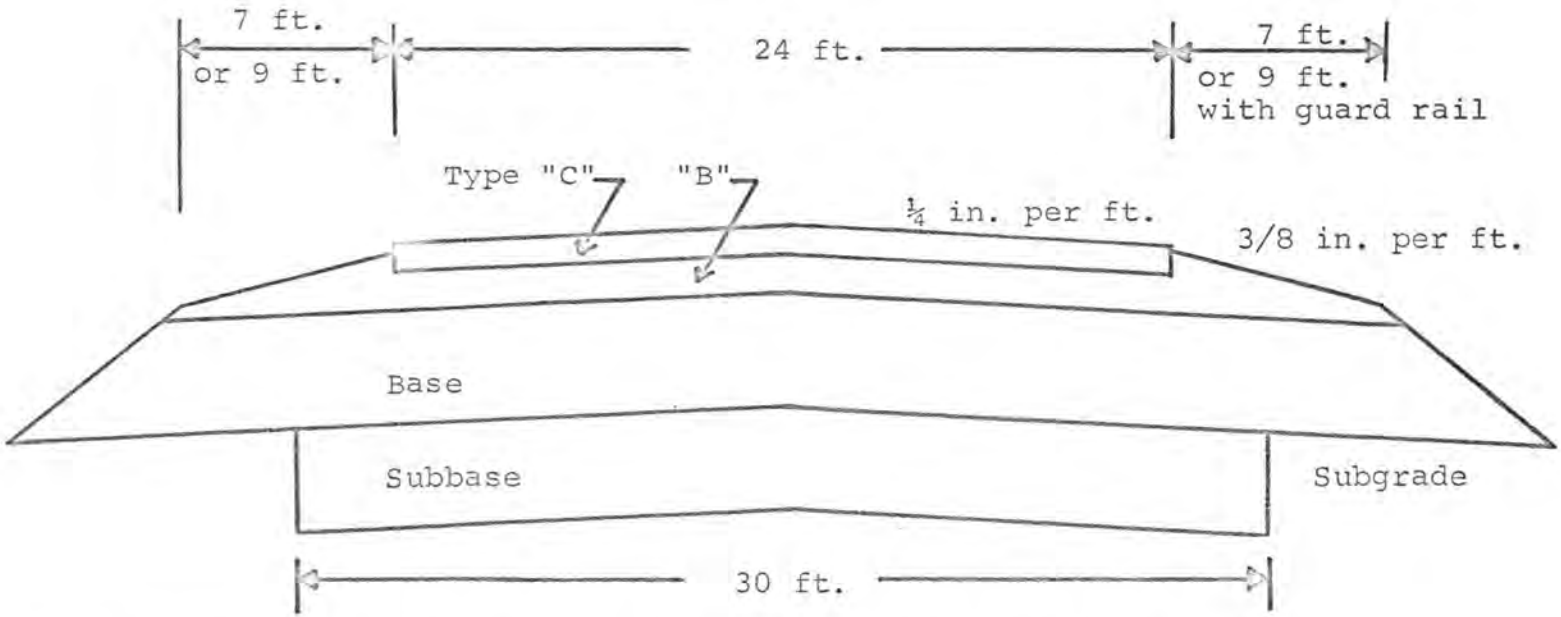
Figure		Page
1	- Typical section on tangent test section.	2
2	- Plot of average deflection versus total thickness of pavement for each type of subbase.	11
3	- Plot of ratio of load to deflection, L/2U, versus total thickness of pavement.	15
4	- Plot of pavement unit strength, L/2UH, versus test date for the inner wheelpaths.	21
5	- Plot of pavement unit strength versus test date for the outer wheelpaths.	22
6	- Plot of ratio of load to deflection versus equivalent thickness, H_e, for April 1963.	35
7	- Plot, same as Figure 6 for April 1965.	37
8	- Plot, same as Figure 6 for May 1966.	38
9	- Plot of normalized unit strength, $L/2UH_e$, versus test date. Precipitation and freezing index are also shown.	48
10	- Plot of computed subgrade modulus versus percent length of test section on fill.	54
11	- Logarithmic plot of average deflections versus estimated years service of test section.	68

INTRODUCTION

This report presents an evaluation of the performance of a flexible pavement on Route 169, Gentry County in Northwest Missouri. The pavement was built in 1961-62, to evaluate the effects of phosphoric acid and lime stabilization of a heavy clay soil. The evaluation of this project was included in Study Number 63-1 Missouri Road Test AASHO Satellite Program when it was initiated in 1963. The construction phase of this project was reported in 1965. See reference (1).

The pavement consists of 3 in. of asphaltic concrete on two thicknesses (4 and 8 in.) of Type 1 aggregate (rolled stone) base placed upon four types of subbase: (a) natural soil, (b) 5 in. by 30 ft. of natural soil modified by 2.8 percent of phosphoric acid, (c) 5 in. by 30 ft. of natural soil modified by six percent of hydrated lime, and (d) 5 in. by 30 ft. of Type 1 aggregate. See Figure 1 for a typical cross section and Table 1 for the physical characteristics of the roadway. A more complete description appears in reference (1).

As constructed, each of the sixteen test sections was 660 feet long. The design thickness for the first set of eight sections was 3-8-0, and 3-8-5 in. The design thickness for the second set of eight sections was 3-4-0, and 3-4-5 in. Locations on the roadway were selected at random, within each set, for the duplicate test sections with each



Asphaltic Concrete: 1¼ in. type "C" and 1-3/4 in. type "B".
Base: 4 in. or 8 in. Type 1 Aggregate.
Subbase: None or 5 in. Acid or Lime stabilized soil, or 5 in. Type 1 Aggregate.
Shoulder: Seal Coat.

FIGURE 1 - TYPICAL SECTION ON TANGENT

Table I - PHYSICAL CHARACTERISTICS OF THE ROADWAY

Type "C" Asphaltic Concrete, 1.25 in.	
Percent Compaction	98.5
Percent Voids	6.1
Direct Compressive Strength, psi.	368
Type "B" Asphaltic Concrete, 1.75 in.	
Percent Compaction	100.0
Percent Voids	4.7
Direct Compressive Strength, psi.	331
Type I Aggregate Subbase, 4 and 8 in.	
Optimum Moisture, %	7.4
Percent Moisture	3.3 - 7.0
Percent Compaction	92.8 - 103.5
Maximum Density, pcf.	135.7 - 140.3
Plastic Index	3.3 - 6.6
Subgrade	
Soil Type, Shelby	
Soil Texture, Clay	
Classification, A-7-6	
Group Index	13 - 20
Optimum Moisture, %	14.4 - 18.0
Percent Moisture	14.4 - 20.1
Percent Compaction	94.3 - 101.8
Maximum Density, pcf.	106.8 - 118.8
Plastic Index	28.5 - 37.7
Laboratory CBR	3.6
Hydrated Lime Treated Subbase, 5 in.	
Lime added was 6% of dry weight of soil	
Optimum Moisture, %	20.7 - 25.3
Percent Moisture	22.6 - 26.4
Percent Compaction	96.0 - 97.2
Maximum Density, pcf.	95.4 - 101.1
Plastic Index	6.0
Laboratory CBR	71.7
Direct Compressive Strength, psi. at 7 days,	More than 50
Ortho-Phosphoric Acid Treated Subbase, 5 in.	
(75% H_3PO_4 in an aqueous solution)	
Acid added was 2.8% of dry weight of soil except for test section 14 where an additional 1.5% was used to neutralize the calcium carbonate content of the soil.	
Optimum Moisture	14.0 - 20.0
Percent Moisture	12.8 - 19.6
Percent Compaction	86.7 - 96.7
Maximum Density, pcf.	103.6 - 114.9
Plastic Index	8.0 - 16.0
Laboratory CBR	74.4
Direct Compressive Strength, psi. at 7 days,	15 - 55

type of subbase. Tests and observations were confined to the central 600 feet of each test section.

Most of the data available for evaluating the performance of these sections consisted of Benkelman beam deflection measurements, and annual samples of load and traffic variables. Only a limited amount of data was obtained for calculating present serviceability index (PSI) as our CHLOE profilometer was not obtained until 1965.

Considerable time was spent in evaluating these data because the relatively small amount provided an excellent opportunity to develop procedures which are expected to be potential guidelines for evaluating data being accumulated on numerous satellite test sections on other projects. Consequently, the importance of the scope of this report exceeds that of evaluation of the data obtained on the Gentry County project.

FINDINGS

Attention is directed to the fact that the objective evaluation of the acid and lime treated subbases was obtained by analysis of variance. The model used for analysis assumes that the indications apply only to the Gentry County project. Also the procedures developed in this report for computing values of (a) equivalent thickness of pavement, (b) subgrade modulus, (c) equivalent thickness required to restrict deflections to a tolerable level, (d) tolerable deflection for a particular thickness of asphaltic concrete, and (e) estimated years service, should presently be considered as tentative.

With these restrictions the following indications appear justified:

1. One inch of rolled stone subbase is approximately equivalent to five inches of phosphoric acid treated subbase.
2. Two inches of rolled stone subbase is approximately equivalent to five inches of lime treated subbase.
3. The inner wheelpath is stronger than the outer wheelpath, and the difference of about ten percent when built increased to about fifty percent after two years service, because of the greater loss of strength in the outer wheelpath.
4. There is very little difference between spring and fall values of pavement strength. Within the inner wheelpaths, the difference was not significant. A small real difference was obtained for the outer wheelpaths.
5. The equivalent thickness of all test sections as built was inadequate to restrict deflections to a tolerable level in the outer wheelpaths after two years service. (Tentative tolerable deflections for various applications of equivalent 18 kip axle load were obtained by an approximate mathematical conversion of the State of California's tentative tolerable deflections for various applications of equivalent 5000 lb. wheel load.)

6. Computed values of subgrade modulus were obtained which appear reasonable, and indicate no difference between cut and fill in the outer wheelpaths. The values of subgrade modulus for fill were greater than the values for cut in the inner wheelpaths.
7. The data were not sufficient to determine any correlation between the change in pavement strength and amount of precipitation or freezing index.
8. A one-point determination of present serviceability index tended to rate the subbases in the same order as that obtained by deflection measurements, but of course, a one-point determination cannot be as discriminatory as a sequence of measurements.
9. There appears to be a definite relationship between pavement deflections and expected service life that merits additional study.

PRESENTATION AND DISCUSSION OF DATA

Benkelman beam normal* deflection measurements were begun in October 1962, soon after construction was completed, and continued two or three times per year through

---

* The beam probe was inserted between the rear dual tires to a point about four feet ahead of the axle. After the measurements for this project were begun, this procedure was modified on other projects by placing the beam probe one foot ahead of the axle. Axle loads varied from 13.9 to 16.5 kip for the years 1962, 1963, and 1964. In 1965 and 1966 an axle load of 17.9 kip was used. Tabulated values of deflection in this report are extrapolated for an axle load of 18.0 kip.

1966. Two days were required to obtain one set of deflection measurements. Seven tests were made at intervals of 100 ft. in each wheelpath in both the northbound and southbound traffic lanes. Tables 2A and 2B show the results for inner and outer wheelpaths, as averages of 14 values for each test section at each of the different times, except that the values for date 8-65 excluded three extremely large deflections within test section No. 12 (.207 and .097 in. respectively for the southbound and northbound inner wheelpaths, and .203 in. for the southbound outer wheelpath). Also the values for date 5-66 excluded three extremely large deflections. (.253 in. for the inner wheelpath of test section No. 12; and .194 in. and .217 in. for the outer wheelpaths of test section No. 13.)

A study of the average deflections shown in Tables 2A and 2B indicates that:

1. For the same type subbase, the deflections of the thicker pavements were less than the deflections of the thinner pavements.
2. The deflections in the outer wheelpaths were predominately greater than those in the inner wheelpaths.
3. The smallest deflections were obtained on test sections with the rolled stone subbase, and generally the largest deflections were obtained on the test sections with no subbase.
4. The deflections obtained on test sections with lime treated subbase were generally less than

Table 2A

AVERAGE VALUES OF BENKELMAN BEAM NORMAL DEFLECTIONS FOR FIRST SET OF TEST SECTIONS WITH 8 INCH BASE

Subbase	None		Acid		Lime		Rolled Stone	
Design	3-8-0		3-8-5		3-8-5		3-8-5	
Test Sec.	5	7	2	8	1	3	4	6
H, in.(a)	11	11	16	16	16	16	16	16
Date	Inner Wheelpaths: .001 in.							
10-62	18	19	17	23	16	18	15	13
4-63	27	26	22	26	21	20	19	15
8-63	24	25	24	25	18	20	17	12
10-63	22	22	23	22	17	18	17	12
4-64	24	24	26	23	18	17	16	12
7-64	27	26	31	26	20	23	22	14
11-64	22	20	23	21	20	19	18	13
4-65	32	27	37	22	27	24	23	14
8-65	31	26	35	25	28	29	23	15
5-66	31	20	33	20	25	23	22	15
10-66	31	21	32	22	28	25	22	17
Avg.(b)	27	24	29	23	22	22	20	14
Avg.(c)	25.4		25.9		22.0		16.9	
	Outer Wheelpaths: .001 in.							
10-62	20	21	22	26	18	21	15	13
4-63	26	27	29	27	24	23	18	15
8-63	23	25	33	25	24	24	20	13
10-63	24	23	27	21	19	22	18	14
4-64	38	29	33	23	28	24	24	18
7-64	44	33	45	26	39	37	33	23
11-64	29	24	29	22	26	23	23	18
4-65	58	49	45	28	41	34	39	37
8-65	44	34	45	29	39	35	30	30
5-66	47	28	47	24	38	30	31	25
10-66	48	33	38	28	39	31	28	29
Avg.(b)	38	30	37	25	32	28	26	22
Avg.(c)	34.3		31.2		30.0		24.3	

(a) H = Total thickness of pavement.
(b) Test section average, excluding data for 10-62.
(c) Type subbase average.

Table 2B

AVERAGE VALUES OF BENKELMAN BEAM NORMAL DEFLECTIONS FOR SECOND SET OF TEST SECTIONS WITH 4 INCH BASE

Subbase	None		Acid		Lime		Rolled Stone	
Design	3-4-0		3-4-5		3-4-5		3-4-5	
Test Sec.	12	13	14	15	9	11	10	16
H, in.[a]	7	7	12	12	12	12	12	12
Date	Inner Wheelpaths: .001 in.							
10-62	24	21	26	23	24	26	18	21
4-63	36	35	35	35	27	30	23	26
8-63	38	41	35	34	28	33	22	25
10-63	32	35	29	29	24	33	20	24
4-64	35	39	28	26	25	34	20	23
7-64	43	50	33	31	28	44	20	24
11-64	32	40	25	25	21	28	19	21
4-65	53	76	34	38	26	42	22	30
8-65	38[d]	49	32	32	29	46	21	26
5-66	43[d]	56	27	25	22	37	19	23
10-66	46	52	34	32	24	41	20	28
Avg.[b]	40	47	31	31	25	37	21	25
Avg.[c]	43.8		31.0		31.1		22.8	
	Outer Wheelpaths: .001 in.							
10-62	30	26	34	30	25	31	19	22
4-63	36	38	41	44	29	33	22	26
8-63	42	46	46	43	30	43	21	26
10-63	36	38	40	37	23	33	19	25
4-64	37	46	38	38	28	43	23	31
7-64	48	65	50	49	35	57	27	36
11-64	35	46	40	36	23	35	21	27
4-65	62	92	57	66	41	53	33	52
8-65	51[d]	66	63	53	43	66	31	38
5-66	62	70[d]	45	49	33	51	27	38
10-66	52	60	60	53	39	47	32	43
Avg.[b]	46	57	48	47	32	46	26	34
Avg.[c]	51.4		47.4		39.2		29.9	

(a) H = Total thickness of pavement.
(b) Test section average, excluding data for 10-62.
(c) Type subbase average.
(d) Average of 12 or 13 deflections.

those obtained on comparable sections placed on natural soil, and greater than those obtained on comparable sections placed on rolled stone subbase. This indicates that some benefit was obtained with the lime treatment.

5. The deflections obtained on test sections with acid treated subbase were generally greater than those obtained on comparable sections with lime treated subbase, and were generally less than those obtained on comparable sections with no subbase; except that the deflections in the inner wheelpath were about the same for two comparisons, (a) the 3-4-5 in. design of the acid and lime treated sections in Table 2B, and (b) the 3-8-5 in. acid treated sections, and 3-8-0 in. sections in Table 2A. This indicates that some benefit was obtained with the acid treatment, but the benefit was less than that provided by the lime treatment.

A plot of the average deflections versus total thickness, for each type of subbase is shown in Figure 2. The curves represent the relationship between these variables for the four pavement designs where Type 1 aggregate was used exclusively beneath the 3 in. of asphaltic concrete surface. In these four designs the thicknesses of Type 1 aggregate were 4, 8, 9, and 13 in. This plot illustrates the greater deflection in the outer wheelpath, the decrease in deflection with increased thickness, and the greater deflections for

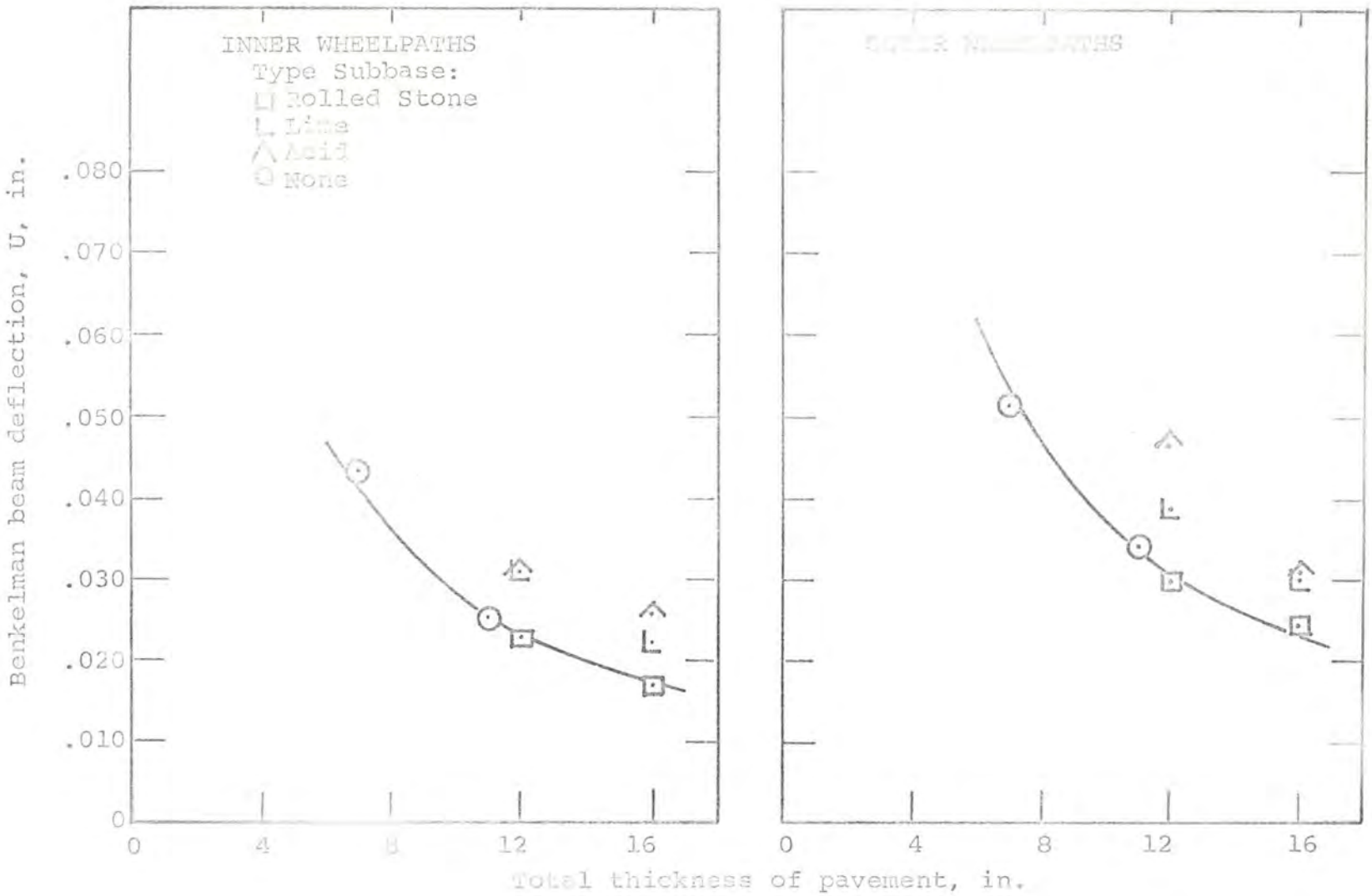


FIGURE 2 - AVERAGE BENKELMAN BEAM DEFLECTIONS VERSUS TOTAL THICKNESS OF PAVEMENT FOR EACH TYPE OF SUBBASE

acid and lime treated subbases than for the same thicknesses of all stone subbase.

It is apparent from the curves in Figure 2 that the relationship between deflection and thickness is not linear, and that the reduction in deflection per inch increase in thickness becomes progressively less with increase in pavement thickness. The curves are equilateral hyperbolas, drawn according to the assumption that the product of the deflection and the total thickness will have a constant value for a particular situation. Accordingly a transformation of the data by use of the reciprocal of either one of the variables should linearize the relationship; the reciprocal of the deflection is selected, and the mathematical model adopted for analysis of the deflection data is introduced:

When Poisson's ratio is 0.5, the Boussinesq relation for the elastic deformation of the subgrade beneath a horizontal plane (which is below the surface of a homogenous mass) and beneath the center of a loaded flexible circular plate on the surface, is shown on page 24 of reference (2) as:

$$\Delta = \frac{3pa^2}{2E\ (a^2 + z^2)^{\frac{1}{2}}} \qquad (1A)$$

where: Δ = elastic deformation, in.,

p = pressure on plate, psi.,

a = plate radius, in.,

E = subgrade modulus, psi., and

z = the thickness of the layer above the horizontal plane, in.

When the subgrade is paved, a similar relation may be written for the deflection of the pavement surface under load if the small compression of the pavement is omitted. Assuming also that half of an axle load (wheel load) is equivalent to πpa^2 the equation becomes:

$$U = \frac{3000\ L}{4\pi E\ (a^2 + H^2)^{\frac{1}{2}}} \qquad (1B)$$

where: U = deflection, in.,

L = axle load, kip, and

H = z = pavement thickness, in.

Deflections computed by equation (1B) would be considerably larger than have been reported in the literature for flexible pavement, therefore, the equation is modified by including a relative stiffness factor, $N^{1/3}$, for the ratio of the stiffness of the pavement to the stiffness of the subgrade as suggested in reference (3).

$$U = \frac{3000\ L}{4\pi EH\ (1 + (a/H)^2)^{\frac{1}{2}}\ N^{1/3}} \qquad (2)$$

where: $N = E_p/E$, and

E_p = pavement modulus, psi.

The equivalent of radius "a" is about 6.2 in. for a 9 kip wheel load on tires inflated to 75 psi, and for "H" greater than "a" the first approximation of relation (2) becomes:

$$U = \frac{1000\ A\ L}{2EH(N)^{1/3}} \qquad (3)$$

where $A = \dfrac{3}{2\pi(1 + (a/H)^2)^{\frac{1}{2}}}$ and

is considered as a constant in this report. A transformation of equation (3) yields the adopted model:

$$\frac{L}{2UH} = \frac{E\ (N)^{1/3}}{1000\ A} \qquad (4)$$

The term L/2UH is called the pavement unit strength and includes all the data which will later be used for the first analysis of variance. Tests to determine values for E and N were not made, but the values should be reasonably uniform in one kind of wheelpath for one set of deflection measurements. A transformation of data is sometimes made to stabilize the variance, however, the transformation is made here because of the assumption that U should be in the denominator in order to have additivity of effects. (Bennett and Franklin state on page 352 of reference (4), "..., but it should be noted that a transformation which stabilizes the variance may be of little value if the additivity of the effects is prejudiced.")

A plot, of average values of L/2U versus total thickness of pavement for each type of subbase is shown in Figure 3. It is apparent that the relationship is practically linear for the four designs with Type 1 aggregate (the sections with no subbase, and with rolled stone subbase), and that an intercept of zero is appropriate for the data. The slope of the line is proportional to the subgrade modulus as shown by equation (4). Assuming that the subgrade modulus for the acid treated and lime treated test sections is the same as that for the other sections, then it is appropriate to draw a parallel line to represent the acid treated sections and another parallel line to represent the lime treated sections. Such parallel lines have

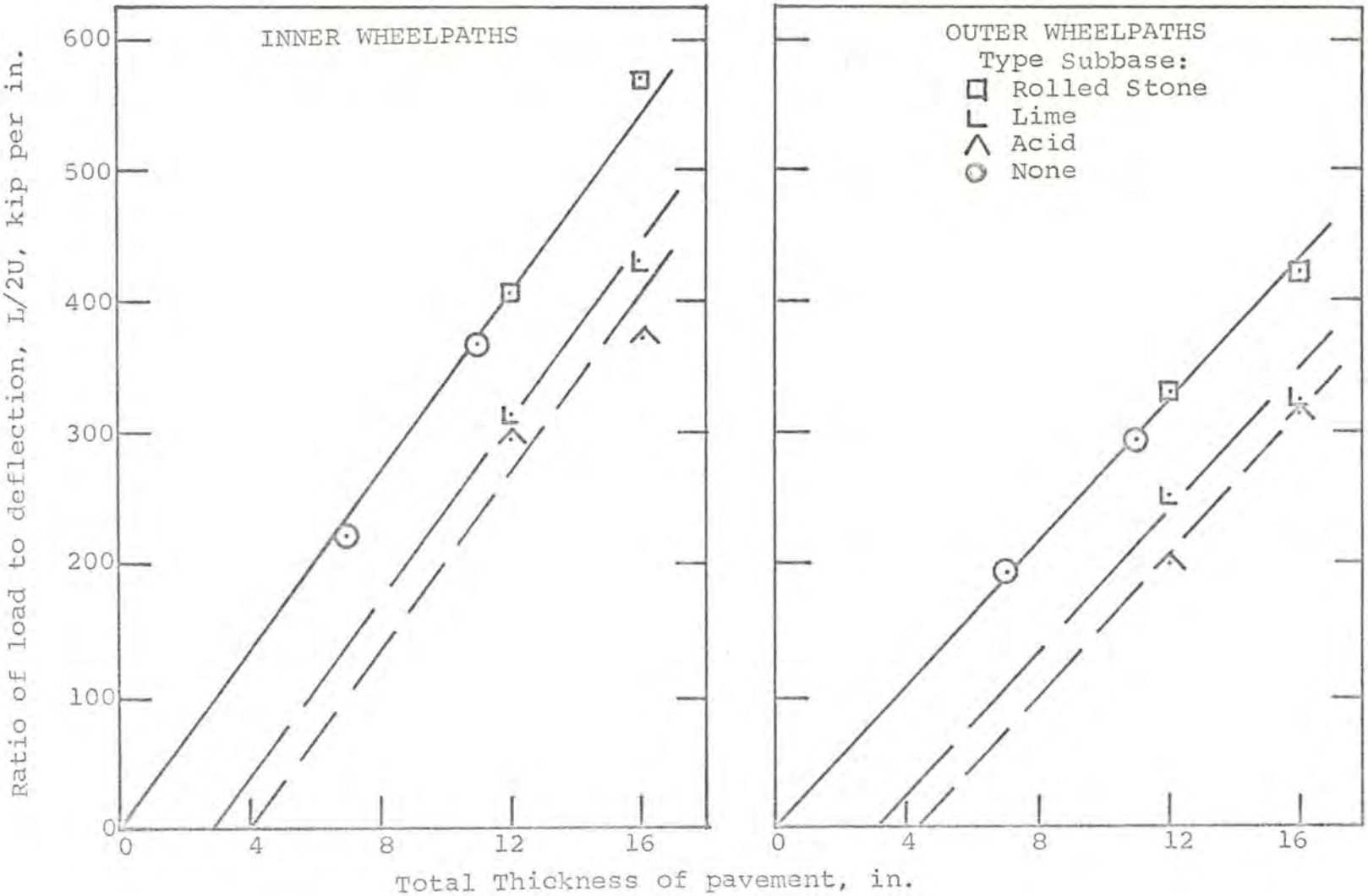


FIGURE 3 - RELATION OF AVERAGE VALUES OF L/2U WITH TOTAL THICKNESS OF PAVEMENT (All values for 1963, 1964, 1965, and 1966 were averaged for each type-subbase design.)

positive intercepts on the thickness axis, and the intercept values provide an indication of the deficiencies of the five inch thickness of treated subbases when compared with rolled stone subbase.

Tables 3A and 3B show numerical values for the averages of L/2U used for constructing Figure 3. In addition, individual averages for each date, wheelpath, and test section are shown. The values tabulated here are not directly obtainable from the average values of fourteen deflections shown in Tables 2A and 2B; since the method for computation of L/2U was, first to average the seven deflections for each date-lane-wheelpath for each test section, and then to multiply the reciprocal of that value by the wheel load, L/2. Such reciprocal values are used for all subsequent computations. Later, it will be shown by analysis of variance that there is no significant difference between traffic lanes in the inner wheelpaths, and that there is only a small difference between lanes in the outer wheelpaths, accordingly the individual values shown in Tables 3A and 3B are average values for two lanes.

Values of the pavement unit strength, L/2UH, were then computed for each date-lane-wheelpath of each test section. The total thickness, H, was considered to be the design thickness. The results of the computations are shown in Tables 4A and 4B as average values for two traffic lanes. The trends of L/2UH for the pavements with each type subbase are illustrated in Figure 4 for the inner wheelpath, and in Figure 5 for the outer wheelpath.

Table 3A

AVERAGE RATIOS OF LOAD PER IN. DEFLECTION, L/2U, FOR FIRST SET OF TEST SECTIONS WITH 8 IN. BASE

Subbase	None		Acid		Lime		Rolled Stone	
Design	3-8-0		3-8-5		3-8-5		3-8-5	
Test Sec.	5	7	2	8	1	3	4	6
H, in.(a)	11	11	16	16	16	16	16	16
Date	Inner Wheelpaths: kip per in.							
10-62	500	470	550	400	560	510	620	720
4-63	330	350	410	350	440	450	490	600
8-63	380	360	380	360	500	450	550	750
10-63	420	420	400	420	550	500	550	750
4-64	380	380	350	400	520	530	560	750
7-64	340	350	300	350	450	400	410	670
11-64	420	450	390	440	460	490	500	720
4-65	280	340	240	420	330	380	400	640
8-65	290	350	260	360	320	320	390	600
5-66	290	450	270	450	360	400	410	600
10-66	290	440	280	420	320	360	420	550
Avg.(b)	342	389	328	397	425	428	468	663
Avg.(c)	366		362		426		566	
	Outer Wheelpaths: kip per in.							
10-62	450	430	420	350	520	440	600	720
4-63	350	330	310	330	380	390	500	620
8-63	400	360	270	360	380	380	460	720
10-63	380	390	340	430	480	420	500	650
4-64	240	320	270	400	330	380	380	520
7-64	210	270	200	350	230	250	280	430
11-64	310	380	310	420	350	390	400	520
4-65	160	190	200	330	220	270	230	250
8-65	210	270	200	320	230	260	300	310
5-66	190	320	190	380	240	310	290	370
10-66	190	270	240	330	230	300	330	320
Avg.(b)	264	310	253	365	307	335	367	471
Avg.(c)	287		309		321		419	

(a) H = Total thickness of pavement.
(b) Test section average, excluding data for 10-62,
(c) Type subbase average.

Table 3B

AVERAGE RATIOS OF LOAD PER IN. DEFLECTION, L/2U
FOR SECOND SET OF TEST SECTIONS WITH 4 IN. BASE

Subbase	None		Acid		Lime		Rolled Stone	
Design	3-4-0		3-4-5		3-4-5		3-4-5	
Test Sec.	12	13	14	15	9	11	10	16
H, in.[a]	7	7	12	12	12	12	12	12
Date	Inner Wheelpaths: kip per in.							
10-62	380	430	350	390	380	350	500	430
4-63	250	260	260	260	330	300	400	350
8-63	240	220	260	260	330	270	410	370
10-63	280	260	310	320	390	270	460	380
4-64	260	230	330	350	360	270	450	390
7-64	210	180	280	290	320	210	450	380
11-64	280	230	360	360	430	320	490	430
4-65	170	120	270	240	350	220	410	310
8-65	240	180	290	280	320	200	430	350
5-66	210	160	340	360	420	250	490	400
10-66	200	180	270	280	380	220	450	330
Avg.[b]	234	202	297	300	363	253	444	369
Avg.[c]	218		298		308		406	
	Outer Wheelpaths: kip per in.							
10-62	310	350	270	300	360	290	470	420
4-63	250	240	220	210	310	280	420	350
8-63	220	200	200	210	300	210	430	350
10-63	250	240	230	250	390	270	470	370
4-64	240	200	240	240	330	210	390	290
7-64	190	140	180	180	260	160	330	250
11-64	260	200	230	250	390	260	430	330
4-65	150	100	160	140	220	170	280	180
8-65	180	140	140	170	210	140	290	240
5-66	150	130	200	180	270	180	340	240
10-66	180	150	150	170	230	190	290	210
Avg.[b]	207	174	195	200	291	207	367	281
Avg.[c]	190		198		249		324	

(a) H = Total thickness of pavement.
(b) Test section average, excluding data for 10-62.
(c) Type subbase average.

Table 4A

AVERAGE VALUES OF PAVEMENT UNIT STRENGTH, L/2UH
FOR FIRST SET OF TEST SECTIONS WITH 8 IN. BASE

Subbase	None		Acid		Lime		Rolled Stone	
Design	3-8-0		3-8-5		3-8-5		3-8-5	
Test Sec.	5	7	2	8	1	3	4	6
H, in.[a]	11	11	16	16	16	16	16	16
Date	Inner Wheelpaths: kip per in. per in.							
10-62	45	43	34	25	35	32	39	39
4-63	30	32	25	23	27	28	30	38
8-63	34	33	24	22	32	28	34	47
10-63	38	38	25	27	34	31	34	47
4-64	34	35	23	25	32	33	35	47
7-64	31	32	19	23	28	25	26	41
11-64	38	41	25	27	29	30	31	45
4-65	26	31	15	26	21	25	25	40
8-65	27	31	16	23	20	20	25	37
5-66	27	41	17	28	23	25	26	38
10-66	26	41	18	26	20	23	26	35
Avg.[b]	31	36	21	25	27	27	29	42
Avg.[c]	33.3		22.8		26.7		35.4	
Set Average				29.6				
	Outer Wheelpaths: kip per in. per in.							
10-62	41	39	26	22	32	28	37	45
4-63	31	30	19	21	23	25	31	39
8-63	36	33	18	22	25	23	29	45
10-63	34	36	22	27	29	27	32	41
4-64	23	28	18	25	22	24	24	33
7-64	19	25	13	22	14	16	18	27
11-64	28	35	19	27	22	24	25	33
4-65	15	16	13	20	14	17	15	16
8-65	19	24	13	20	14	16	19	20
5-65	17	29	13	23	15	19	18	23
10-66	18	25	16	20	15	19	20	20
Avg.[b]	24	28	16	23	19	21	23	30
Avg.[c]	26.0		19.6		20.1		26.4	
Set Average				23.0				

(a) H = Total thickness of pavement.
(b) Test section average, excluding data for 10-62.
(c) Type subbase average.

Table 4B

AVERAGE VALUES OF PAVEMENT UNIT STRENGTH, L/2UH
FOR SECOND SET OF TEST SECTIONS WITH 4 IN. BASE

Subbase	None		Acid		Lime		Rolled Stone	
Design	3-4-0		3-4-5		3-4-5		3-4-5	
Test Sec.	12	13	14	15	9	11	10	16
H, in.[a]	7	7	12	12	12	12	12	12
Date	Inner Wheelpaths: kip per in. per in.							
10-62	54	61	30	32	31	29	42	36
4-63	36	37	21	22	28	25	33	30
8-63	34	31	22	22	28	22	34	30
10-63	40	38	26	26	32	27	38	32
4-64	37	33	28	29	30	22	37	33
7-64	30	26	23	24	27	18	37	32
11-64	40	33	30	30	36	27	40	36
4-65	25	17	22	20	30	18	35	25
8-65	34	26	24	23	26	17	36	29
5-66	35	24	28	30	35	20	41	33
10-66	28	26	22	23	32	19	38	28
Avg.[b]	34	29	25	25	30	22	37	31
Avg.[c]	31.5		24.8		25.9		33.8	
Set Average			29.0					
	Outer Wheelpaths: kip per in. per in.							
10-62	45	50	22	25	30	24	40	35
4-63	36	34	19	17	26	23	35	29
8-63	31	28	16	18	25	17	36	30
10-63	36	35	19	21	33	22	40	31
4-64	35	28	19	20	28	18	33	25
7-64	27	20	15	16	21	14	28	22
11-64	37	28	19	21	33	22	36	28
4-65	21	14	13	12	18	14	23	15
8-65	26	20	12	14	17	12	24	20
5-65	21	19	17	15	23	15	29	20
10-66	25	21	13	14	20	16	24	18
Avg.[b]	29	25	16	17	24	17	31	24
Avg.[c]	27.1		16.5		20.8		27.3	
Set Average			22.9					

(a) H = Total thickness of pavement.
(b) Test section average, excluding data for 10-62.
(c) Type subbase average.

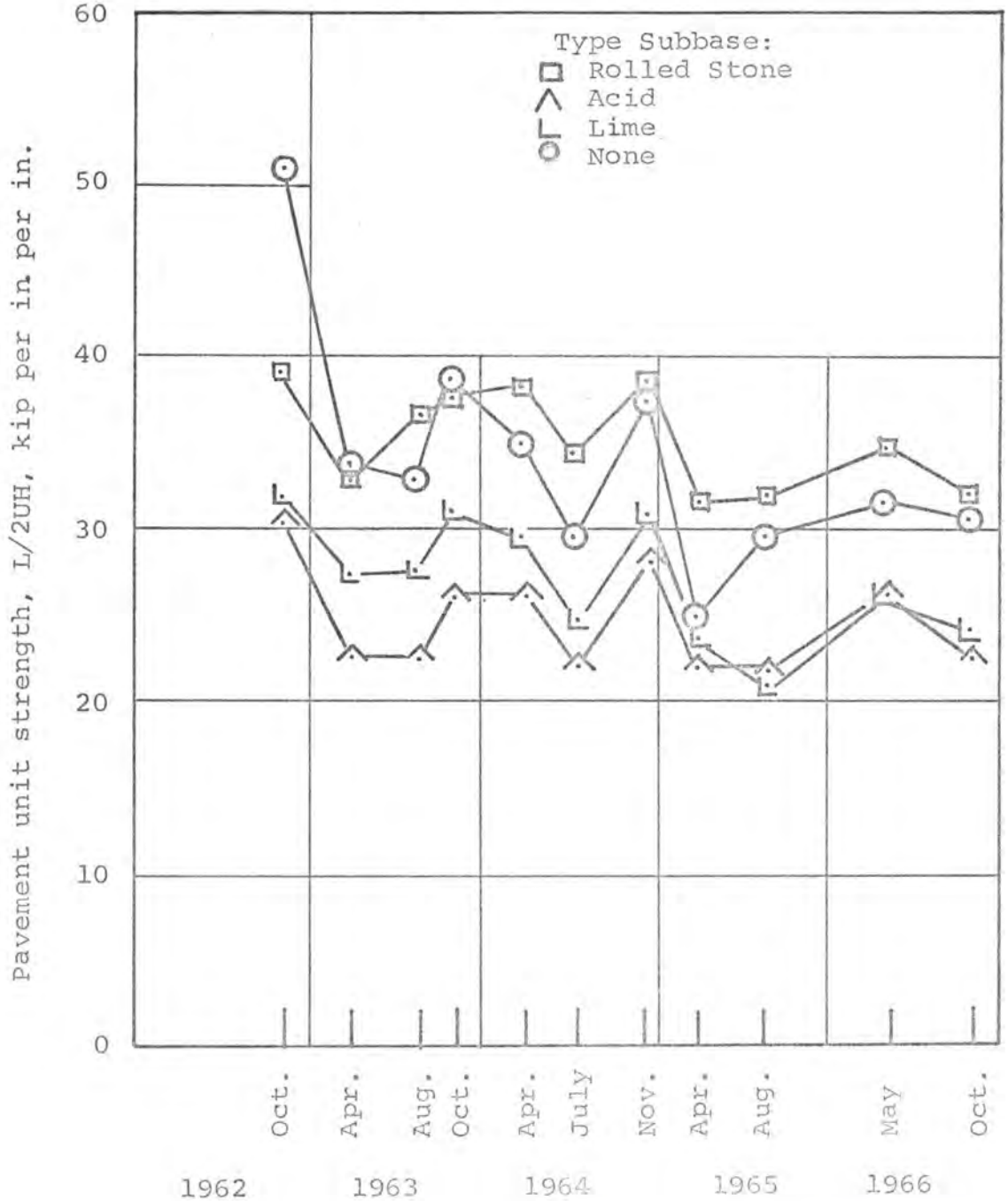


FIGURE 4 - RELATION OF PAVEMENT UNIT STRENGTH WITH TEST DATE FOR THE INNER WHEELPATHS ON EACH TYPE SUBBASE

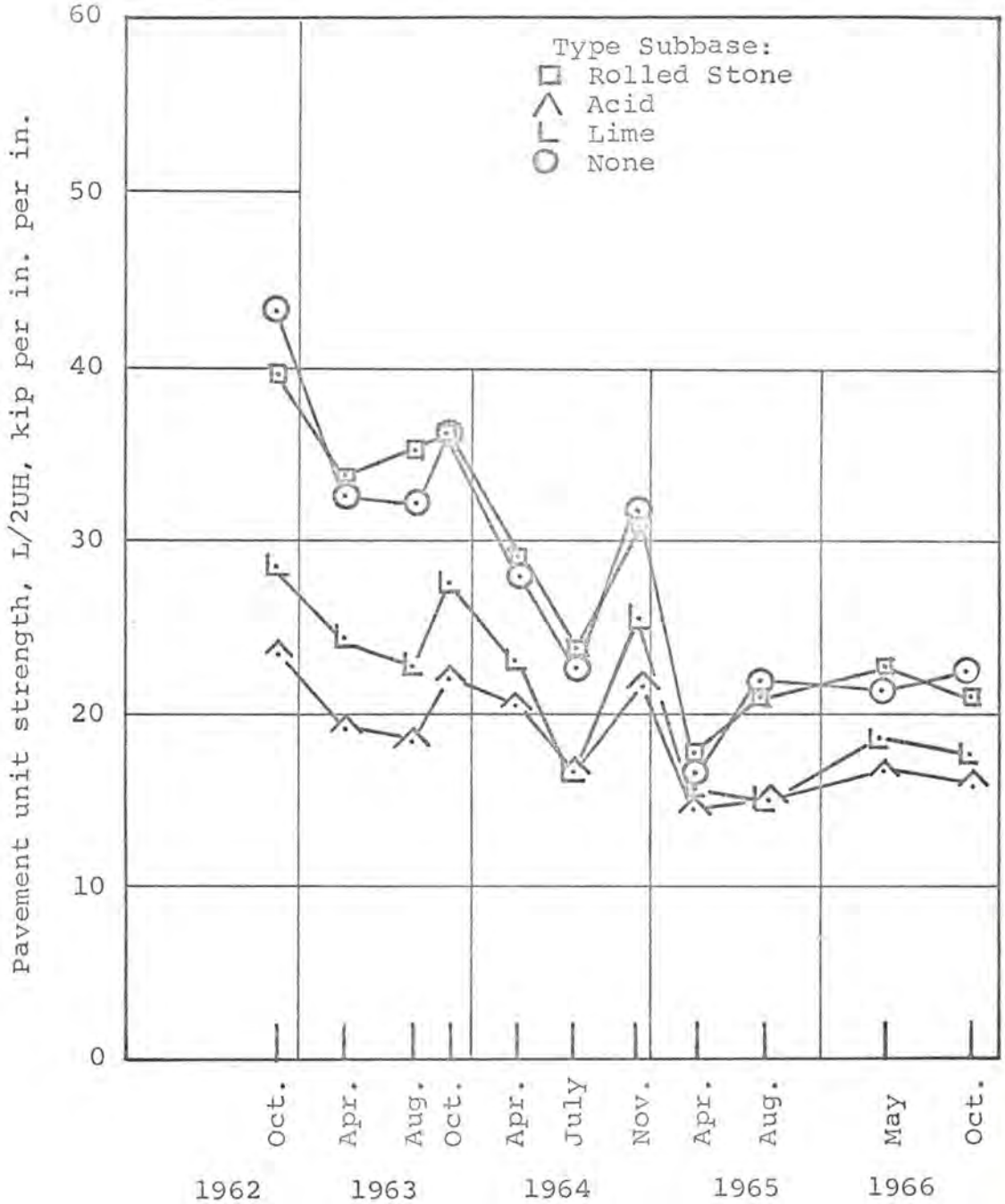


FIGURE 5 - RELATION OF PAVEMENT UNIT STRENGTH WITH TEST DATE FOR THE OUTER WHEELPATHS ON EACH TYPE SUBBASE

A study of average values for pavement unit strength indicates that:

1. The pavement unit strength is greater in the inner wheelpath than it is in the outer wheelpath.
2. Within wheelpaths, there is practically no difference between the two set averages for 8 in., and 4 in. base and this indicates that the term, L/2UH, is sufficient to explain the effect of thickness.
3. Within wheelpaths, there is very little difference in unit strength of the four designs where Type 1 aggregate was the only material used for base, and subbase; also all of the type subbase averages for these designs are greater than the set averages.
4. Within wheelpaths, the average values for acid treated, and lime treated subbases are less than the set averages, and this indicates that the treated subbases have less strength than that of an equal thickness of rolled stone subbase.
5. Within wheelpaths, the unit strength of each test section for date 7-64 is less than the unit strength for 4-64, and for 11-64; except the inner wheelpath of test section No. 10 where the summer unit strength is equal to the spring unit strength. This indicates that larger deflections, and corresponding lower unit strengths, may occur for the higher summer temperatures, because of some additional compression of the asphaltic surface.

6. Within wheelpaths, the unit strength for each type of subbase is greatest for October 1962, the date of the first Benkelman beam deflections. Figure 4 indicates a small loss of unit strength after two years for the inner wheelpaths. Figure 5 shows a considerable loss of unit strength after two years for the outer wheelpaths.

All of the indications stated above and the indications stated earlier for deflections are subjective since a criterion for estimating the degree of significance of the different values shown in Tables 2 and 4 has not yet been established.

ANALYSIS OF VARIANCE OF L/2UH

Analysis of variance, an effective statistical tool, provides an objective method for separating causative effects from a complex array of data, and provides statistical inferences for making conclusions with a known risk of making an incorrect conclusion.

Table 5 shows the results of analyses of variance of the pavement unit strength, L/2UH, for: 2 replicate test sections, 2 traffic lanes, 2 pavement thicknesses, 4 types of subbase, 4 years, and 2 seasons; also the two kinds of wheelpath. The analysis is simpler for the same number of seasons each year, therefore, the data for dates 10-62, 7-64, and 8-63 were excluded. The remaining seasons are called spring and fall seasons, presuming that the data for date 8-65 may represent a fall season. Notation for

Table 5

ANALYSES OF VARIANCE OF PAVEMENT UNIT STRENGTH, L/2UH
1963 TO 1966 SPRING AND FALL VALUES

Source	DF	SS	MS	Ratio	F	pf	s
Inner wheelpaths: (Avg. strength = 29.4 kip per in. per in.)							
Y years	3	2065	688	22.2	5.8	.001	3.2
S Seasons	1	75	75	2.4	3.9	.05	0.6
R Subbase	3	4632	1544	49.8	5.8	.001	4.9
H Thickness	1	1	1	0.0	3.9	.05	0.0
L Lanes	1	31	31	1.0	3.9	.05	0.0
YS	3	361	120	3.9	3.9	.01	1.7
YR	9	234	26	0.8	2.0	.05	0.0
RH	3	137	46	1.5	2.7	.05	0.0
LH	1	86	86	2.8	3.9	.05	0.9
II	102	1554	15	0.5	1.4	.05	0.0
error	128	4092	31	1.0			5.6
Total	255	13268					
pool	230	5646	25	-			5.0
Outer wheelpaths: (Avg. strength = 22.9 kip per in. per in.)							
Y	3	5944	1981	99.0	5.8	.001	5.5
S	1	153	153	7.6	6.8	.01	1.0
R	3	3225	1075	53.7	5.8	.001	4.1
H	1	2	2	0.1	3.9	.05	0.0
L	1	248	248	12.4	11.4	.001	1.3
YS	3	111	37	1.8	2.7	.05	0.0
YR	9	597	66	3.3	3.3	.001	1.7
RH	3	246	82	4.1	3.9	.01	1.4
LH	1	124	124	6.2(a)	3.9	.05	1.3
II	102	855	8	0.6	1.4	.05	0.0
error	128	2555	20	1.0			4.5
Total	255	14060					
pool	230	3410	15	-			3.9
From six-way analysis of variance:							
W Wheelpath	1	5520	5520	212.3	11.1	.001	4.6
PE Interactions with all main effects	254	20681	81	3.1	1.4	.001	-
error	256	6647	26				5.1
Total	511	32848					

(a) Becomes significant at .01 level if pool is used for test.

Table 5 is shown in Table 6. The sum of squares for each source effect was computed from an appropriate subtotal of the data. The subtotals used are not shown, but most of them can be obtained approximately from Tables 4A and 4B. The values for average mean square are obtained by dividing the sum of squares by the respective degrees of freedom. The ratio is obtained by dividing the average mean square of each source effect by the mean square for error. The level of significance of the source effect is then determined by comparing the value of its mean square ratio with standard values which are published as values of F in most textbooks on statistical theory. The value of F can be found in tables if three elements are known: (a) the degrees of freedom of the source, (b) the degrees of freedom of the residual error, and (c) the level of significance of the test.

The test for level of significance is based upon the null hypothesis that the source effects do not differ significantly. To reject the null hypothesis when it is actually true is to commit an error of the first kind. To accept the null hypothesis as true when it is actually false is known as an error of the second kind. The customary procedure is to select a small value for "P_f", the probability of occurrance of an error of the first kind. The numerical value selected is known as the level of significance of the test. Bennett and Franklin suggest the following values and descriptive terms on page 149, reference (4).

Table 6

NOTATION AND COMPONENTS OF AVERAGE MEAN SQUARE FOR TABLES 5 AND 10

DF = Degrees of freedom.
SS = Sum of squares of residuals from average value.
MS = Average mean square (SS/DF).
Ratio = Ratio of two mean squares (two variances).
F = Standard ratio of two variances for the degrees of freedom and probability shown.
p_f = Probability level of significance of the test.
s = Estimated standard deviation.

H_e = Equivalent thickness levels in Table 10.
V(x) = Individual variance component, V(W) is the variance component of kind of wheelpath, etc.

Main effects	Components of Avg. Mean Square
W = Between kinds of wheelpath.	V(e) + 256 V(W)
Y = Between years.	V(e) + 64 V(Y)
S = Between seasons.	V(e) + 128 V(S)
R = Between types of subbase.	V(e) + 64 V(R)
H = Between thickness levels.	V(e) + 128 V(H)
L = Between traffic lanes.	V(e) + 128 V(L)
Interactions	
YS = Years - seasons.	V(e) + 32 V(YS)
YR = Years - Type subbase.	V(e) + 16 V(YR)
RH = Type subbase - thickness.	V(e) + 32 V(RH)
LH = Traffic lane - thickness.	V(e) + 64 V(LH)
II = Insignificant.	V(e) + zero
error (e) = Residual error (replicates).	V(e)

pool = A pool of error item and insignificant interactions.
PE = A pool of all interactions and main effects except W.

The number in the average mean square above is the number of values for each subtotal of a source effect.

1. Not significant when the mean square ratio is less than F for $p_f = 0.05$.
2. Of questionable significance, when the ratio is between the values of F for $p_f = 0.05$ and for $p_f = 0.01$.
3. Significant, when the ratio exceeds the value of F for $p_f = 0.01$. (Less than one chance in 100 that the source effect is not significant.)

The equations for the variance components of the average mean square are shown in Table 6. These equations are based on an analysis of variance model which assumes that the indications derived apply only to the Gentry County project. The principal variables are not random, since they were preselected for years, seasons, types of subbase, and pavement thickness. Consequently, the mean square for residual error becomes the denominator of all the mean square ratios. When the mean square ratio has a value less than two and is not significant, the variance component for the source effect is considered to be zero. (See page 392 of reference 4, or page 415 of reference 5.) Items conforming to this rule provide additional estimates of the residual error, e.g., note the equation for insignificant interactions in Table 6. Some people would pool the degrees of freedom and sum of squares for such items with those for replicates to obtain a better estimate of the residual error mean square, and would use the pooled value for computing the ratios for tests of significance level. A "non-pooler" would use the replicate mean square for computing the ratios.

K. A. Brownlee states on page 415 of reference (5) "The never-pooler can be confident that his errors of the first kind have the stated probability. The sometimes-pooler may be somewhat uncomfortable about this, but he will claim that his errors of the second kind have smaller probability than those of the never-pooler. If the sample sizes are large so that degrees of freedom are plentiful, the motivation or temptation to be a sometimes-pooler are less."

Degrees of freedom are plentiful for the error item in Table 5, it is for replicates alone and was used for the denominator of the mean square ratios shown. A value for the pool of replicates and insignificant interactions is also shown; and if that is used for the denominator, the ratios become 24 percent larger in the inner wheelpath, and 33 percent larger in the outer wheelpath. Only one evaluation of significance would be changed in Table 5, however, the traffic lane - thickness interaction in the outer wheelpath would go from nonsignificant to significant. The following descriptive terms for the level of significance brackets have been selected for use in this report.

1. Insignificant, when the mean square ratio is less than two and is not significant.
2. Not significant or nonsignificant, when the ratio is less than the value of F for $p_f = 0.01$. This is indicated in Tables 5 and 10 by a value of 0.05 for p_f.
3. Significant, when the ratio is between the values of F for $p_f = 0.01$ and for $p_f = 0.001$. This is

indicated in Tables 5 and 10 by a value of 0.01 for p_f.

4. Highly significant, when the ratio exceeds the value of F for $p_f = 0.001$. (Less than one chance in 1000 that the source effect is not significant.) This is indicated in Tables 5 and 10 by a value of 0.001 for p_f.

Each analysis in Table 5 for the inner and outer wheelpaths is a five-way crossed analysis of variance which separates the contributions of five sources to the variance. There are 26 interactions; 10 two-factor, 10 three-factor, 5 four-factor, and 1 five-factor. The mean square of each two-factor interaction was computed, but only those which were not insignificant in one or the other wheelpath are recorded. The three-factor, four-factor, and five-factor interactions were assumed to be insignificant since their average mean square from 88 degrees of freedom was less than half that for replicates. These were pooled with the balance of the two-factor interactions, and are shown as insignificant interactions, II. Another analysis, not fully shown, for a six-way crossed analysis of variance also included the kind of wheelpath, and indicated that the interaction of wheelpath with years was highly significant. (The six-way analysis for the data in Table 10 indicated that all of the two-factor interactions of kind of wheelpath with the other sources were highly significant.) This indicates that the change of strength with time is different in the two kinds of wheelpath, and that additional analyses

should be simpler and more informative if the data for the two kinds of wheelpath are examined separately. The recapitulation of the six-way analysis at the bottom of Table 5 shows a very large mean square ratio (212.3) for the difference between wheelpath strengths, it is many times larger than the ratio of 11.1 which would indicate that this source effect is highly significant.

Generally the analysis of variance is concluded when the levels of significance have been determined, but in this report values have also been computed for the individual standard deviations. The standard deviation is equal to the square root of the variance component and provides additional information, e.g., the contribution of seasons to the variance of the outer wheelpaths is indicated in table 5 to be significant, however, the value of 1.0 for its standard deviation is less than five percent of the average strength, and is considerably less than the standard deviation of 4.5 for replicates. On the other hand the contribution of years is indicated as highly significant with a standard deviation of 5.5 which is 24 percent of the average strength, and larger than the standard deviation for replicates. The inference is that the difference of strength between spring and fall seasons is real but small, and that there is an appreciable difference of strength between years in the outer wheelpaths. The same information would be available from the respective subtotals of the effects if they were shown.

Analysis of variance has separated the effects of the principal variables, and it is of particular interest to note that the thickness effect is insignificant, and that its standard deviation is zero. The inference is that the term L/2UH is sufficient to explain the effect of thickness, at least for these data where the difference in thickness levels is only four inches. The term L/2UH, however, does not explain the effect of type of subbase (R) which is highly significant and has a large standard deviation. A method for evaluating the effect of type of subbase is shown below.

EQUIVALENT THICKNESS

It is hypothesized that equation (4) can be modified to include the concept of an equivalent pavement thickness, H_e, with the result:

$$\frac{L}{2\ UH_e} = \frac{E(N)^{1/3}}{1000\ A} = K \qquad (5)$$

where H_e = the equivalent thickness of the pavement, in., and

K = normalized unit strength, kip per in. deflection per in. equivalent thickness.

Herein, it is also assumed that, for one set of deflection measurements in one kind of wheelpath:

1. E is approximately the same for all test sections,
2. N is practically a constant,
3. A is practically a constant,
4. K is approximately a constant; and is equal to the average value of pavement unit strengths, L/2UH, for the test sections with no subbase, and with rolled stone subbase, and

5. Equation (5) is suitable for evaluating effects that influence Benkelman beam deflections.

Then an equivalent thickness may be computed as:

$$H_e = L/2UK = (L/2UH)(H/K) \qquad (6)$$

This assumes that if the pavement had been built with three inches of asphaltic concrete over sufficient rolled stone to provide a total thickness of H_e, the performance of such pavement, as measured by Benkelman beam deflections, should be the same as the performance of the real test sections.

The upper portion of Table 7 shows averages for eight values of the unit strength, L/2UH, in each wheelpath for each year, 1963 to 1966. (Two seasons, two lanes, and two replicate test sections provide eight values.) The last column, for average K, is the average value for the designs with no subbase, and with rolled stone subbase. Values of equivalent thickness are shown in the lower portion of Table 7, as computed by equation (6) for each value tabulated in the upper portion. The average values for all wheelpaths were then adjusted to the nearest inch, considering the condition that for one kind of subbase treatment, the quantity $H-H_e$ should be a constant.

Figure 6 shows a plot of values of L/2U for each test section versus the equivalent thickness, H_e, for April 1963. The use of "H_e" for the abscissa, instead of the real thickness, H, in effect shifts the values for test sections with acid treated, and lime treated subbase to the left; and is apparent if Figure 6 is compared with Figure 3.

Table 7

VALUES OF THE EQUIVALENT THICKNESS,
COMPUTED FROM AVERAGE VALUES OF PAVEMENT UNIT STRENGTH

Subbase	Acid		Lime		Rolled Stone		None		Avg.
Design	3-4-5,	3-8-5	3-4-5,	3-8-5	3-4-5,	3-8-5	3-4-0,	3-8-0	last
H, in.	12	16	12	16	12	16	7	11	4 Col.
Year	Pavement Unit Strength, L/2UH, kip per in. per in. K								
	Inner Wheelpaths:								
1963	24	25	28	30	33	37	38	35	35.8
1964	29	25	29	31	37	40	36	37	37.5
1965	22	20	23	21	31	32	26	29	29.5
1966	26	23	26	23	35	31	28	34	32.0
	Outer Wheelpaths:								
1963	19	22	26	26	34	36	35	33	34.5
1964	20	22	25	23	30	29	32	28	29.8
1965	13	16	15	15	21	17	20	18	19.0
1966	15	18	18	17	23	21	22	22	22.0
	Equivalent Thickness, H_e = (L/2UH)(H/K), in.								
	Inner Wheelpaths:								
1963	8.0	11.2	9.4	13.4	11.1	16.5	7.4	10.8	
1964	9.3	10.7	9.3	13.2	11.8	17.1	6.7	10.9	
1965	8.9	10.8	9.4	11.4	12.6	17.4	6.2	10.8	
1966	9.8	11.5	9.8	11.5	13.1	15.5	6.1	11.7	
Avg.	9.0	11.0	9.5	12.4	12.2	16.6	6.6	11.0	
	Outer Wheelpaths:								
1963	6.6	10.2	9.0	12.1	11.8	16.7	7.1	10.5	
1964	8.1	11.8	10.1	12.3	12.1	15.6	7.5	10.3	
1965	8.2	13.5	9.5	12.6	13.3	14.3	7.4	10.4	
1966	8.2	13.1	9.8	12.4	12.5	15.3	7.0	11.0	
Avg.	7.8	12.2	9.6	12.4	12.4	15.5	7.2	10.6	
	All Wheelpaths:								
Avg.	8.4	11.6	9.5	12.4	12.3	16.0	6.9	10.8	
Use,	8	12	9	13	12	16	7	11	

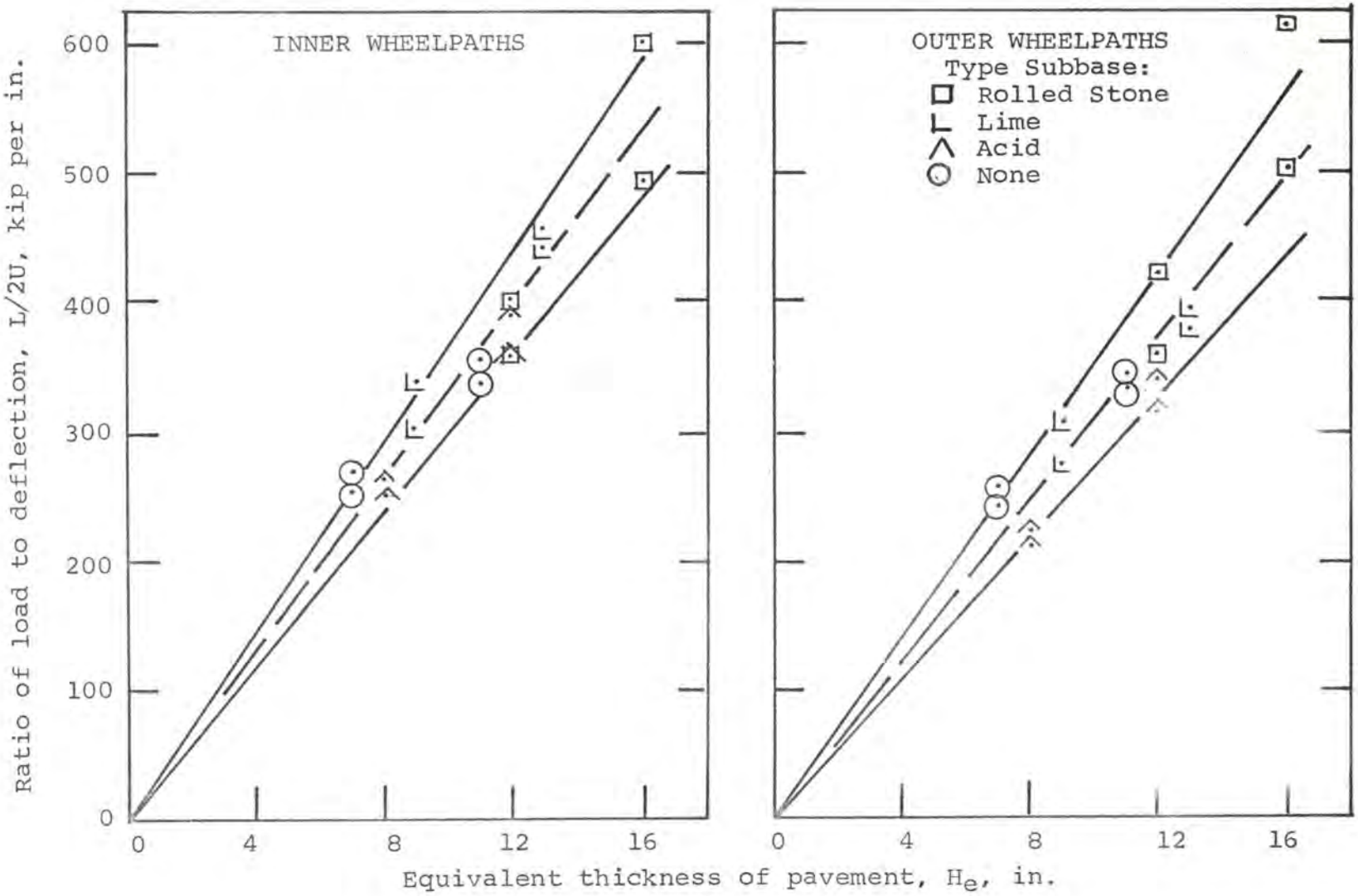


FIGURE 6 - RELATION OF L/2U WITH EQUIVALENT THICKNESS OF PAVEMENT FOR APRIL 1963
(Each point is the average value for a test section.)

The central lines in Figure 6 have the average slopes K_i for the inner wheelpaths, and K_o for the outer wheelpaths. The limiting lines are drawn at plus and minus one standard deviation for the slope. Figures 7 and 8 show similar plots of the data for April 1965 and May 1966 respectively. These figures indicate that: (a) the outer wheelpath is weaker than the inner wheelpath, (b) the strength of both wheelpaths decreases with time, (c) the variation of strength is greater in the inner wheelpath, (d) the variation of strength increases with time, and (e) there may be a recovery of strength, as indicated by comparison of Figure 8 and Figure 7.

Computed values for the subbase equivalent thickness of rolled stone are shown in Table 8. The values were obtained by subtracting seven from the values of H_e, tabulated in Table 7, for the designs 3-4-5 and 3-4-0; and by subtracting eleven from the values of H_e for the designs 3-8-5 and 3-8-0. These remainders are considered to be that thickness of rolled stone which is equivalent to the real thickness of the subbase as built. Random variations from the average values are apparent, as shown by the range (high minus low value) for each column in each wheelpath. Computations for subbase equivalent thickness of rolled stone are not necessary for the designs with rolled stone subbase, and no subbase; but were made to show that their range is of the same order of magnitude as the range for the designs with treated subbase. The averages values for all wheelpaths were adjusted to the nearest inch, considering

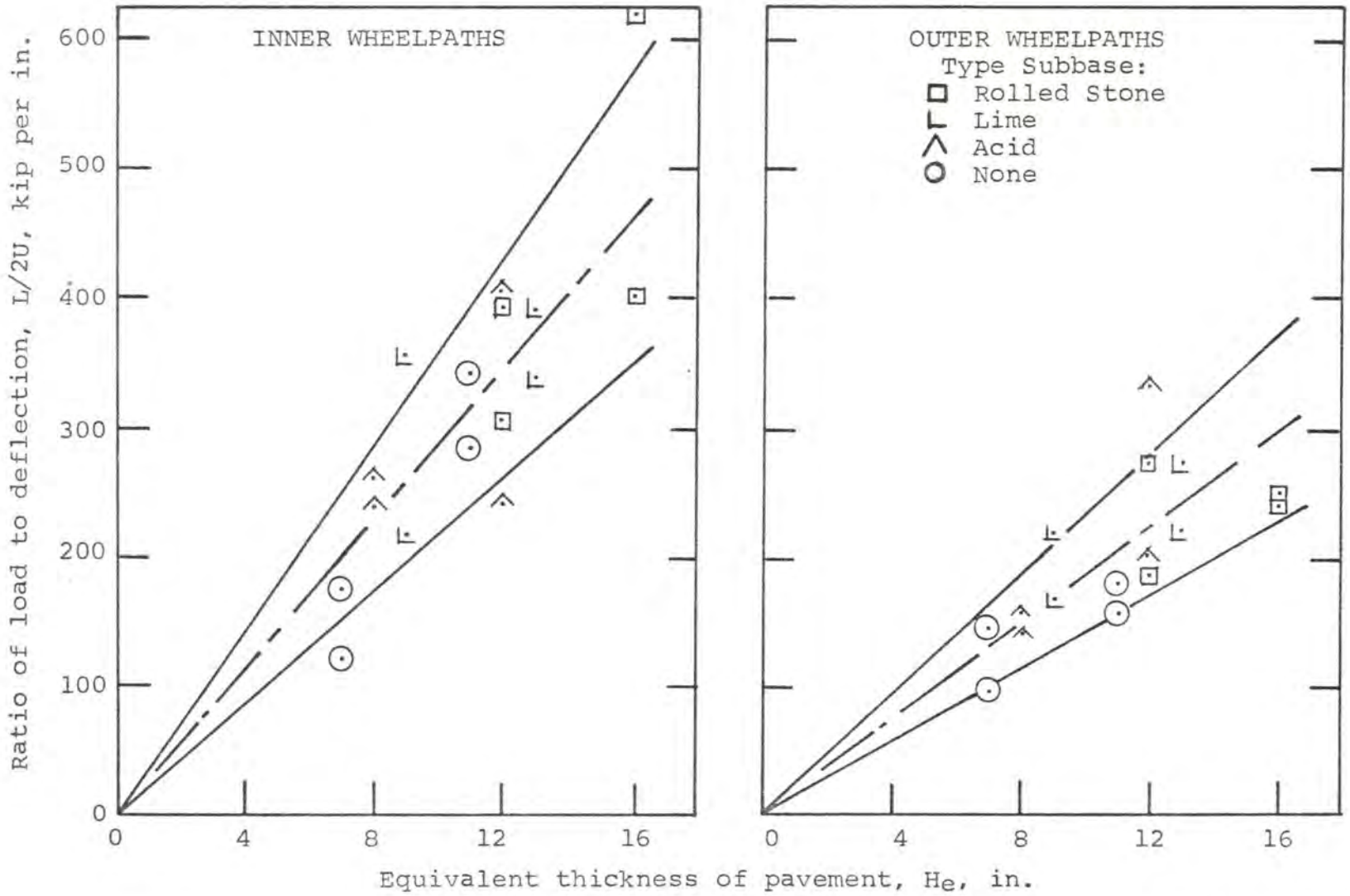


FIGURE 7 - RELATION OF L/2U WITH EQUIVALENT THICKNESS OF PAVEMENT FOR APRIL 1965
(Each point is the average value for a test section.)

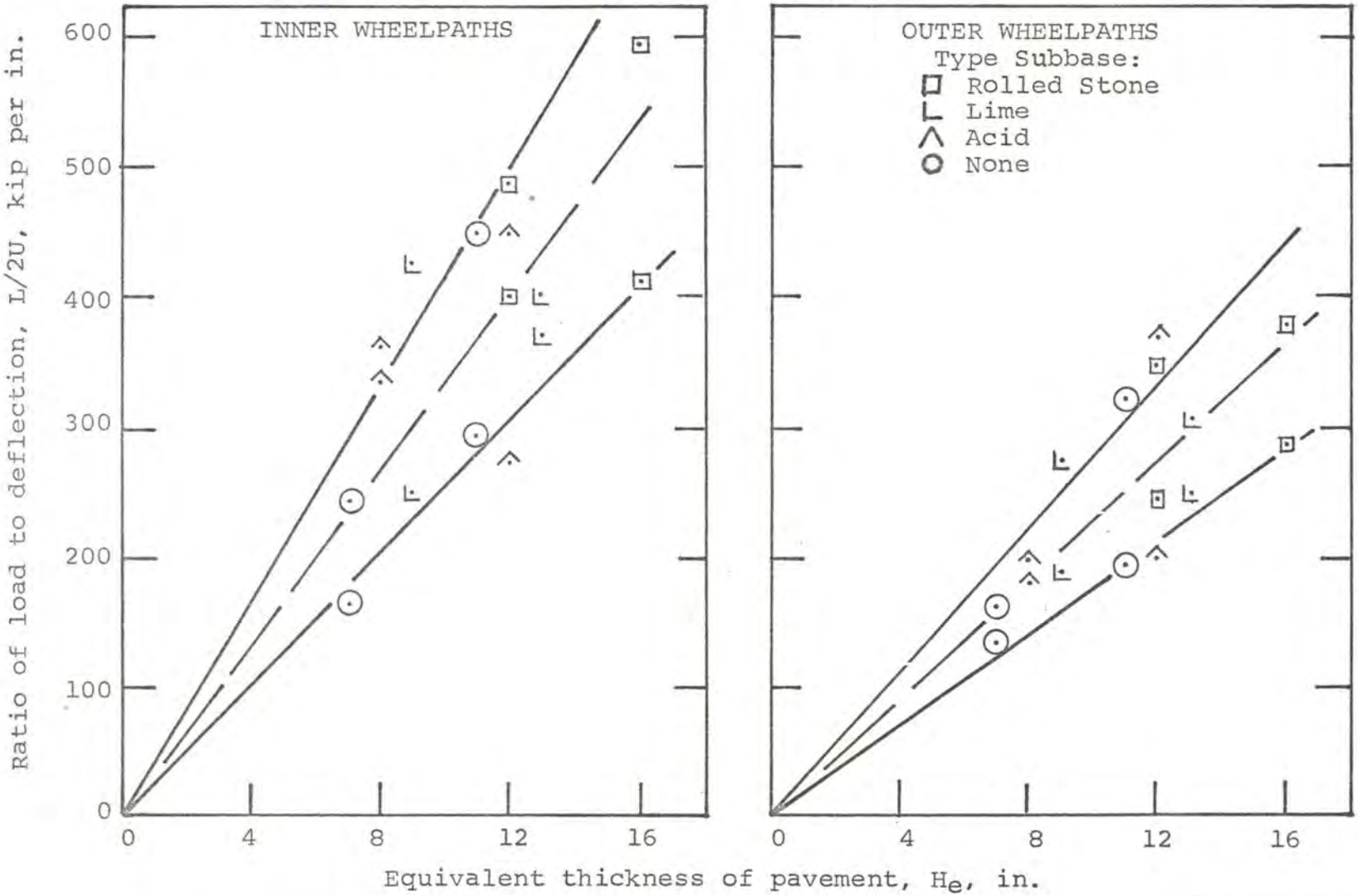


FIGURE 8 - RELATION OF L/2U WITH EQUIVALENT THICKNESS OF PAVEMENT FOR MAY 1966
(Each point is the average value for a test section.)

Table 8

VALUES OF THE SUBBASE EQUIVALENT THICKNESS OF ROLLED STONE

Subbase	Acid		Lime		Rolled Stone		None	
Design	3-4-5, 3-8-5		3-4-5, 3-8-5		3-4-5, 3-8-5		3-4-0, 3-8-0	
H, in.	12	16	12	16	12	16	7	11
H_e,[a] in.	8	12	9	13	12	16	7	11
	Subbase Equivalent, in. Average for spring and fall.							
Year	H_e-7	H_e-11	H_e-7	H_e-11	H_e-7	H_e-11	H_e-7	H_e-11
	Inner Wheelpaths:							
1963	1.0	0.2	2.4	2.4	4.1	5.5	0.4	-0.2
1964	2.3	-0.3	2.3	2.2	4.8	6.1	-0.3	-0.1
1965	1.9	-0.2	2.4	0.4	5.6	6.4	-0.8	-0.2
1966	2.8	0.5	2.8	0.5	6.1	4.5	-0.9	0.7
Avg.	2.0	0.0	2.5	1.4	5.2	5.6	-0.4	0.0
Range	1.8	0.8	0.5	2.0	2.0	1.9	1.3	0.9
	Outer Wheelpaths:							
1963	-0.4	-0.8	2.0	1.1	4.8	5.7	0.1	-0.5
1964	1.1	0.8	3.1	1.3	5.1	4.6	0.5	-0.7
1965	1.2	2.5	2.5	1.6	6.3	3.3	0.4	-0.6
1966	1.2	2.1	2.8	1.4	5.5	4.3	0.0	0.0
Avg.	0.8	1.2	2.6	1.4	5.4	4.5	0.2	-0.4
Range	1.6	3.3	1.1	0.5	1.5	2.4	0.5	0.7
	All Wheelpaths:							
1963	0.3	-0.3	2.2	1.8	4.4	5.6	0.2	-0.4
1964	1.7	0.2	2.7	1.8	5.0	5.4	0.1	-0.4
1965	1.6	1.2	2.4	1.0	6.0	4.8	-0.2	-0.4
1966	2.0	1.3	2.8	1.0	5.8	4.4	-0.4	0.4
Avg.	1.4	0.6	2.5	1.4	5.3	5.0	-0.1	-0.2
Use	1		2		5		0	

(a) Adjusted equivalent thickness selected for use from Table 7.

that for one kind of subbase treatment, the equivalent thickness of rolled stone should be the same for the two designs. These adjusted values based on measurements of pavement deflection indicate that:

1. One inch of rolled stone is equivalent to five inches of natural soil treated with 2.8 percent phosphoric acid, and
2. two inches of rolled stone is equivalent to five inches of natural soil treated with 6 percent hydrated lime.

Values of the normalized strength, $L/2UH_e$, were then computed for each date-lane-wheelpath of the test sections with acid treated, and lime treated subbase by assuming that the equivalent total thickness of the pavement, H_e, was equal to the adjusted values shown in Table 7. The results as average values for two traffic lanes, are shown in Table 9A for the 3-8-5 in. design, and in Table 9B for the 3-4-5 in. design. For comparison Tables 9A and 9B also show the values of $L/2UH_e$ for the test sections with no subbase, and with rolled stone subbase (these values are identical to the values of L/2UH shown in Table 4). The average values for normalized unit strength indicate that, within each kind of wheelpath: (a) there is practically no difference between the set averages for 8 inch and 4 inch base, and (b) there is little difference between the averages for type of subbase. This indicates that the normalized unit strength, $L/2UH_e$, may be sufficient to explain the

Table 9A

AVERAGE VALUES OF NORMALIZED UNIT STRENGTH, $L/2UH_e$
FOR FIRST SET OF TEST SECTIONS WITH 8 IN. BASE

Subbase	None		Acid		Lime		Rolled Stone	
Design	3-8-0		3-8-5		3-8-5		3-8-5	
Test Sec.	5	7	2	8	1	3	4	6
H_e, in.	11	11	12	12	13	13	16	16
Date	Inner Wheelpaths: kip per in. per in.							
10-62	45	43	45	34	43	40	39	39
4-63	30	32	34	30	34	35	30	38
8-63	34	33	32	30	38	35	34	47
10-63	38	38	33	35	42	38	34	47
4-64	34	35	30	33	40	41	35	47
7-64	31	32	25	29	35	30	26	41
11-64	38	41	32	37	36	38	31	45
4-65	26	31	20	35	26	30	25	40
8-65	27	31	22	30	25	24	25	37
5-66	27	41	23	37	28	31	26	38
10-66	26	41	24	35	25	28	26	36
Avg. (a)	31	36	28	33	33	33	29	42
Avg. (b)	33.3		30.3		33.0		35.4	
Set Average	33.0							
	Outer Wheelpaths: kip per in. per in.							
10-62	41	39	35	30	40	34	37	45
4-63	31	30	26	28	29	30	31	39
8-63	36	33	23	30	30	29	29	45
10-63	34	36	29	37	36	33	32	41
4-64	23	28	24	33	26	30	24	33
7-64	19	25	17	29	18	20	18	27
11-64	28	35	26	35	27	30	25	33
4-65	15	16	17	27	17	21	15	16
8-65	19	24	17	26	18	20	19	20
5-65	17	29	17	31	19	23	18	23
10-66	18	25	21	28	18	23	20	20
Avg. (a)	24	28	22	30	24	26	23	30
Avg. (b)	26.0		26.0		24.9		26.4	
Set Average	25.8							

(a) Test section average, excluding data for 10-62.
(b) Type subbase average.

Table 9B

AVERAGE VALUES OF NORMALIZED UNIT STRENGTH, $L/2UH_e$ FOR SECOND SET OF TEST SECTIONS WITH 4 IN. BASE

Subbase	None		Acid		Lime		Rolled Stone	
Design	3-4-0		3-4-5		3-4-5		3-4-5	
Test Sec.	12	13	14	15	9	11	10	16
H_e, in.	7	7	8	8	9	9	12	12
Date	Inner Wheelpaths: kip per in. per in.							
10-62	54	61	44	49	41	39	42	36
4-63	36	37	32	32	38	33	33	30
8-63	34	31	32	33	36	31	34	30
10-63	40	38	39	40	43	36	38	32
4-64	37	33	41	43	40	30	37	33
7-64	30	26	34	36	36	23	37	32
11-64	40	33	45	45	48	36	40	36
4-65	25	17	33	30	39	24	35	25
8-65	34	26	36	35	35	22	36	29
5-66	35	24	42	45	47	27	41	33
10-66	28	26	33	35	42	25	38	28
Avg. (a)	34	39	37	37	40	29	37	31
Avg. (b)	31.5		37.0		34.6		33.8	
Set Average				34.2				
	Outer Wheelpaths: kip per in. per in.							
10-62	45	50	34	37	40	32	40	35
4-63	36	34	27	27	34	30	35	29
8-63	31	28	24	27	33	23	36	30
10-63	36	35	28	31	44	30	40	31
4-64	35	28	30	30	36	24	33	25
7-64	27	20	22	23	29	18	28	22
11-64	37	28	29	31	44	29	36	28
4-65	21	14	20	17	24	19	23	15
8-65	26	20	18	21	24	16	24	20
5-65	21	19	25	23	30	21	29	20
10-66	25	21	19	22	26	22	24	18
Avg. (a)	29	25	24	25	32	23	31	24
Avg. (b)	27.1		24.7		27.8		27.3	
Set Average				26.7				

(a) Test section average, excluding data for 10-62.
(b) Type subbase average.

effect of thickness, and the effect of type of subbase. Another analysis of variance provides an objective test.

ANALYSIS OF VARIANCE OF $L/2UH_e$

The results of analyses of variance of the normalized unit strength, $L/2UH_e$, are shown in Table 10. The principal variables are the same as those for Table 5, except that here there are two levels of equivalent total thickness instead of two levels of real thickness, however, the difference between levels is still four inches (the difference between the sets with base thickness of eight and four inches). Notation for Table 10 is shown in Table 6. The analyses show that the source effect:

1. Kinds of wheelpath, W, is highly significant.
2. Years, Y, is highly significant; and the effect of years is greater in the outer wheelpath than in the inner wheelpath, as shown by the respective mean square ratios, and by the respective individual standard deviations.
3. Seasons, S, is not significant in the inner wheelpaths; but is significant in the outer wheelpaths, however, the value of 1.2 for the standard deviation indicates that the difference between seasons is small.
4. Types of subbase, R, is insignificant in each wheelpath.
5. Equivalent thickness of pavement, H_e, is not significant in the inner wheelpaths, and is not

Table 10

ANALYSES OF VARIANCE OF NORMALIZED UNIT STRENGTH, $L/2UH_e$
1963 TO 1966 SPRING AND FALL VALUES

Source	DF	SS	MS	Ratio	F	p_f	s
Inner Wheelpaths: (Avg. strength = 33.8 kip per in. per in.)							
Y	3	2740	913	22.8	5.8	.001	3.7
S	1	84	84	2.1	3.9	.05	0.6
R	3	131	44	1.1	2.7	.05	0.0
H_e	1	216	216	5.4[a]	3.9	.05	1.2
L	1	41	41	1.0	3.9	.05	0.0
YS	3	485	162	4.0	3.9	.01	2.0
RH_e	3	815	272	6.8	5.8	.001	2.7
LH_e	1	96	96	2.4	3.9	.05	0.9
II	111	2227	20	0.5	1.4	.05	0.0
error	128	5112	40	1.0			6.3
Total	255	11947					
pool	239	7339	31	-			5.5
Outer Wheelpaths: (Avg. strength = 26.2 kip per in. per in.)							
Y	3	7451	2484	95.5	5.8	.001	6.2
S	1	200	200	7.7	6.8	.01	1.2
R	3	40	13	0.5	2.7	.05	0.0
H_e	1	88	88	3.4	3.9	.05	0.7
L	1	333	333	12.8	11.4	.001	1.5
YS	3	162	54	2.1	2.7	.05	0.9
RH_e	3	191	64	2.5	2.7	.05	1.1
LH_e	1	138	138	5.3[a]	3.9	.05	1.3
II	111	1361	12	0.5	1.4	.05	0.0
error	128	3375	26	1.0			5.1
Total	255	13339					
pool	239	4736	20	-			4.4
From six-way analysis of variance:							
W	1	7389	7389	223.9	11.1	.001	5.4
PE	254	16799	66	2.0	1.4	.001	-
error	256	8487	33	1.0			5.7
Total	511	32675					

(a) Becomes significant at .01 level if pool is used for test.

significant in the outer wheelpaths. (If the pool is used for the tests the respective ratios become 7.0 and 4.4 which compared with F = 6.7 for p_f = 0.01 indicate that the source, H_e, is significant in the inner wheelpaths, and is not significant in the outer wheelpaths. The standard deviation in the inner wheelpaths, 1.2, is less than four percent of the average strength and indicates that the difference between equivalent thickness levels is small.)

6. Traffic lanes, L, is insignificant in the inner wheelpaths; but is highly significant in the outer wheelpaths, however, the value of 1.5 for the standard deviation indicates that the average difference between lanes is not large for the outer wheelpaths.
7. Interaction of years and seasons, YS, is significant in the inner wheelpaths, but it is not significant in the outer wheelpaths.
8. Interaction of type of subbase and equivalent thickness, RH_e, is highly significant in the inner wheelpaths, but it is not significant in the outer wheelpaths.
9. Interaction of traffic lane and equivalent thickness, LH_e, is not significant in either wheelpath. (If the pool is used for the tests, the ratios become 3.1 and 6.9 which compared with F = 6.7

for $p_f = 0.01$ indicate that the source, LH_e, is not significant in the inner wheelpaths, but is significant in the outer wheelpaths.)

10. The interaction of years with type subbase, YR, is not recorded separately in Table 10 since it was insignificant, however, in Table 5 it was indicated to be highly significant in the outer wheelpaths.

The purpose of the above analyses, in addition to a separation of the effects of the principal variables, was to determine whether or not the normalized unit strength, $L/2UH_e$, is sufficient to represent the two effects; equivalent thickness, and type of subbase. It is sufficient for the outer wheelpaths, since the effect of type of subbase, the effect of equivalent thickness, and the interaction, RH_e, are all nonsignificant. The expression is also sufficient for R and H_e in the inner wheelpaths, when the test for significance is made by the error for replicates which indicates that the two effects are nonsignificant as shown in Table 10. But if the pool is used for the test, the equivalent thickness becomes significant; however, since the standard deviation is small it can be assumed that $L/2UH_e$ is sufficient to represent the two effects practically but not perfectly. The interaction, RH_e, is highly significant in the inner wheelpaths, and the standard deviation (2.7) is not small. When interactions are significant, various interpretations are possible, one is suggested later when subgrade modulus is discussed. When interactions are non-

significant, the implication is clear that the main effects can be evaluated from the respective totals of each effect.

A concise representation of all the deflection data is shown in Figure 9 as a plot of $L/2UH_e$ versus dates for the inner and outer wheelpaths. In the lower portion of Figure 9 numerical values are also shown for the annual freezing index (degree days), and for the annual freezing period (days between the maximum and minimum values of accumulated degree days). The inches of precipitation as recorded at the nearest weather station for the month preceding the month when deflection measurements were made, are shown by the bar chart. The change in normalized unit strength is illustrated, and is greater for the outer wheelpaths. It may be indicated that the strength will decrease when the freezing index exceeds some value, say 700 degree days. The lower strength for July 1964 could be due to a higher prevailing temperature, or due to the greater precipitation, or both. The annual precipitation varied from 26 to 46 in., and averaged 35 in. for the years 1953 to 1962 inclusive. It was 33 in. for 1963, 37 in. for 1964, and 38 in. for 1965. But it was only 26 in. for 1966, which may account for the recovery of normalized unit strength for that year.

A summary of the normalized-unit-strength averages is shown in Table 11 for each type of subbase and design, and for each wheelpath, and traffic lane. The tabulated averages are for 16 values: two replicate test sections, two seasons, and four years. According to the analysis of

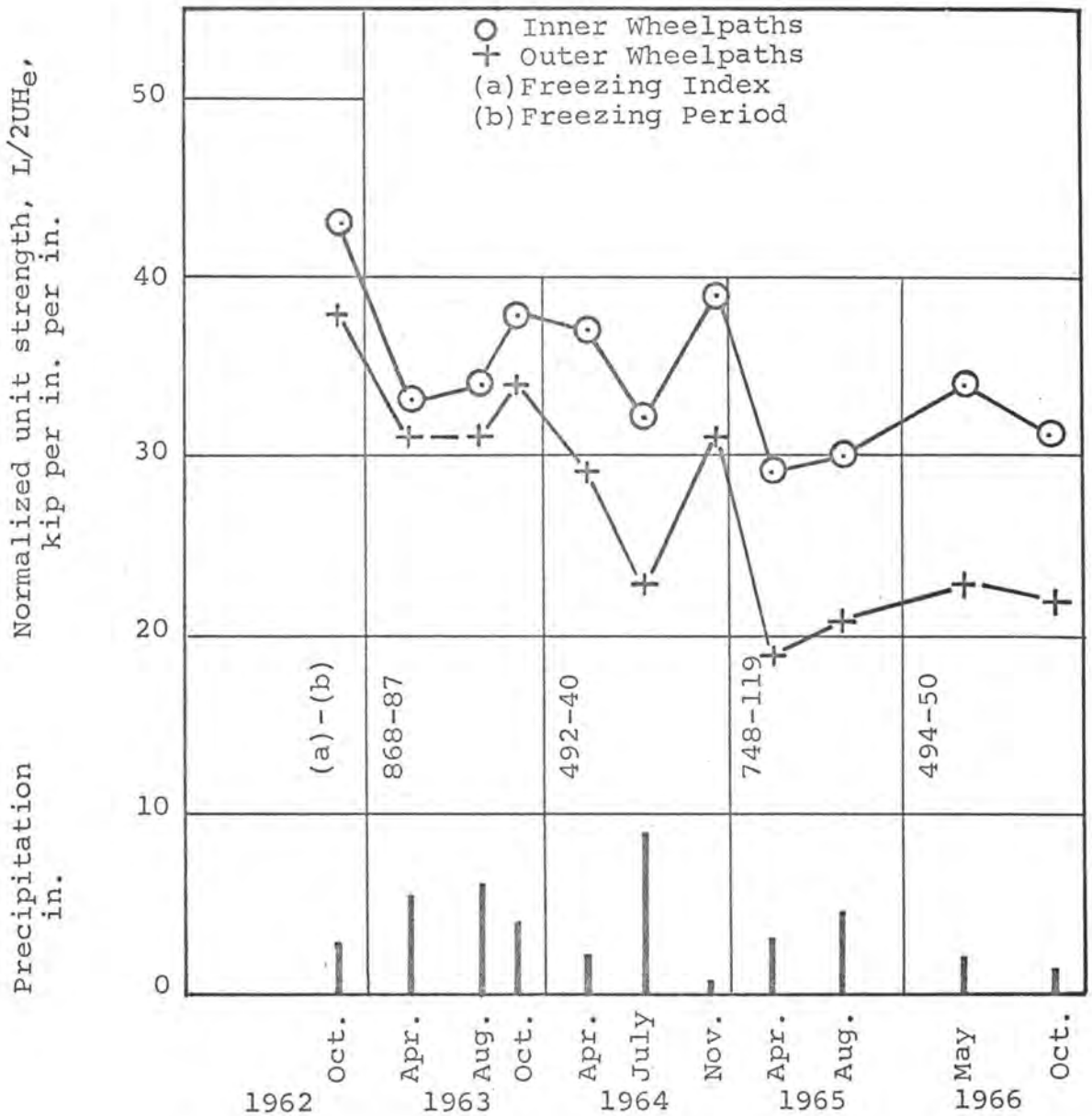


FIGURE 9 - RELATION OF NORMALIZED UNIT STRENGTH OF EACH WHEELPATH WITH TEST DATE
Also shown are precipitation for the preceding month and the freezing index, degree days, for the winter's freezing period, days.

Table 11

SUMMARY OF NORMALIZED UNIT STRENGTH, $L/2UH_e$

Traffic Lane Wheelpath Subbase	Design	H_e, in.	Northbound Inner	Northbound Outer	Southbound Inner	Southbound Outer
None	3-8-0	11	32.4	23.8	34.6	27.2
Acid	3-8-5	12	30.4	24.1	30.8	28.7
Lime	3-8-5	13	31.8	24.0	33.2	26.1
R.S.	3-8-5	16	32.9	23.2	37.1	28.0
First Set Average			31.9	23.8	33.9	27.5
None	3-4-0	7	32.1	26.1	31.5	28.4
Acid	3-4-5	8	36.4	23.7	39.3	26.0
Lime	3-4-5	9	37.2	27.9	33.4	28.0
R.S.	3-4-5	12	34.1	27.9	33.9	25.8
Second Set Average			35.0	26.4	34.5	27.0
Average (two sets)			33.4	25.1	34.2	27.3

SUMMARY OF TRAFFIC VARIABLES

	Northbound Lane			Southbound Lane		
		Equivalent 18 kip[b]			Equivalent 18 kip	
Year	ADT[a]	Cumulated	Annual Avg.	ADT	Cumulated	Annual Avg.
1963	365	9,700	9,700	365	12,000	12,000
1964	385	19,600	9,800	385	24,400	12,200
1965	376	29,700	9,900	394	36,900	12,300
1966	410	40,000	10,000	431	50,800	12,700

(a) Average number of vehicles per day.
(b) Applications equivalent 18 kip axle load.

variance shown in Table 10, the difference between lanes (34.2 - 33.4) is not significant for the inner wheelpaths. The difference between lanes (27.3 - 25.1) for the outer wheelpaths is highly significant for averages of 128 values, it is real but small. According to equation (5) the strength is proportional to the subgrade modulus and the relative stiffness factor, so one inference is that the average subgrade modulus in the southbound outer wheelpath is nine percent greater than that for the northbound outer wheelpath.

Table 11 also shows a summary of traffic variables. Sample counts were made annually to estimate the average daily traffic (A.D.T.), and sample weighings were made to estimate the distribution of truck axle loads. These values were then converted to accumulated applications equivalent 18 kip axle load each direction by use of the fourth power relation. The southbound traffic (18 KE) is 27 percent greater than that northbound. Perhaps the greater southbound traffic load has caused some additional compaction of the subgrade in the outer wheelpath as indicated by the greater southbound normalized unit strength (greater subgrade modulus), but the data from only one test project is not sufficient to establish such a causal relationship.

SUBGRADE MODULUS

Approximate values of the subgrade modulus can be computed by transforming equation (5) to:

$$E = 1000\ AK/N^{1/3} \qquad (7)$$

and assigning a value of 0.41 to A, and a value of 10 to N.

The assigned value of A is an average value obtained by substituting the equivalent thicknesses, H_e, into the relation for A shown with equation (3). The value of 10 for N is frequently used with the Boussinesq relation for deflection of flexible pavement (see page 21 of reference 6). Of couse the actual ratio of pavement modulus to subgrade modulus is not a constant, more than a dozen modifying variables are discussed in reference (7); however, a change of the estimated value of N by a factor of eight only changes the computed value of E by a factor of two in equation (7). Computed values of the subgrade modulus in the inner wheelpaths, and in the outer wheelpaths are shown in Table 12 for eight dates, and for two clases of subbase; (a) the acid treated and lime treated subbase, for which equivalent thicknesses were computed, and (b) the none and rolled stone subbase, for which the real thicknesses were considered to be the equivalent thicknesses. The tabulated averages in Table 12 are for 16 values (two test sections, two lanes, two types of subbase, and two designs).

A study of the computed values for subgrade modulus indicates that:

1. The order of magnitude of the values is reasonable, as shown by comparison with the values reported by A. S. Vesic for the AASHO Road Test, reference (6), that are also shown in Table 12.
2. The values for the outer wheelpaths are appreciably less than the values for the inner wheelpaths. In

Table 12

COMPUTED VALUES FOR SUBGRADE MODULUS, E

	Inner Wheelpaths		Outer Wheelpaths	
	Type Subbase			
Date	Acid and Lime	None and Rolled Stone	Acid and Lime	None and Rolled Stone
4-63	6400	6300 psi	5500	6300 psi
10-63	7300	7300	6400	6800
4-64	7100	6900	5600	5500
11-64	7600	7200	6000	5900
Average	7100	6900	5900	6100
4-65	5600	5300	3900	3200
8-65	5500	5800	3800	4100
5-66	6700	6300	4500	4200
10-66	5900	5900	4300	4100
Average	5900	5800	4100	3900
Spring Avg.	6400	6200	4900	4800
Fall Avg.	6600	6600	5100	5200

The above may be compared with the following values of subgrade modulus for the AASHO Road Test as tabulated on page 18 of reference (6).

BASIS FOR DETERMINATION	SUBGRADE MODULUS
Triaxial compression test by Kansas Highway Dept.	1300 psi
Triaxial compression test by Univ. of California	5500
Van der Poels stiffness factor by Asphalt Institute	3000
Triaxial compression test by Ohio State Univ.	4000
Plate load tests	2040
CBR Tests	2800 to 5600
Measured pavement deflections	7000

1963-64 the ratio of values of OWP/IWP is 0.86, and in 1964-65 the ratio of values of OWP/IWP is 0.69.

3. The values for the last two years are appreciably less than the values for the first two years. In the inner wheelpaths, the decrease was 17 percent; and in the outer wheelpaths, the decrease was 33 percent.
4. There is little difference between the average values for the spring and fall seasons.
5. There is little difference between the values for the two classes of subbase.

The grade for the test sections was mainly a series of cuts and fills in heavily rolling country. Figure 10 shows a plot of the subgrade modulus in each wheelpath versus proportion of length on fill, for each test section. Each point is the average for 16 values (4 years, 2 seasons, and 2 lanes). The line of regression for the inner wheelpaths is:

$$E = 5600 + 1600 \text{ (proportion of length on fill)}, \quad (8)$$

and the correlation coefficient of 0.66 is significant. This indicates that the value of 5600 psi. for the subgrade modulus in cut is 78 percent of the 7200 psi. value for subgrade all on fill. For the outer wheelpaths, the regression was:

$$E = 4700 + 600 \text{ (proportion of length on fill)}, \quad (9)$$

however, since the correlation coefficient of 0.33 was non-significant for 16 plotted points, the line of regression

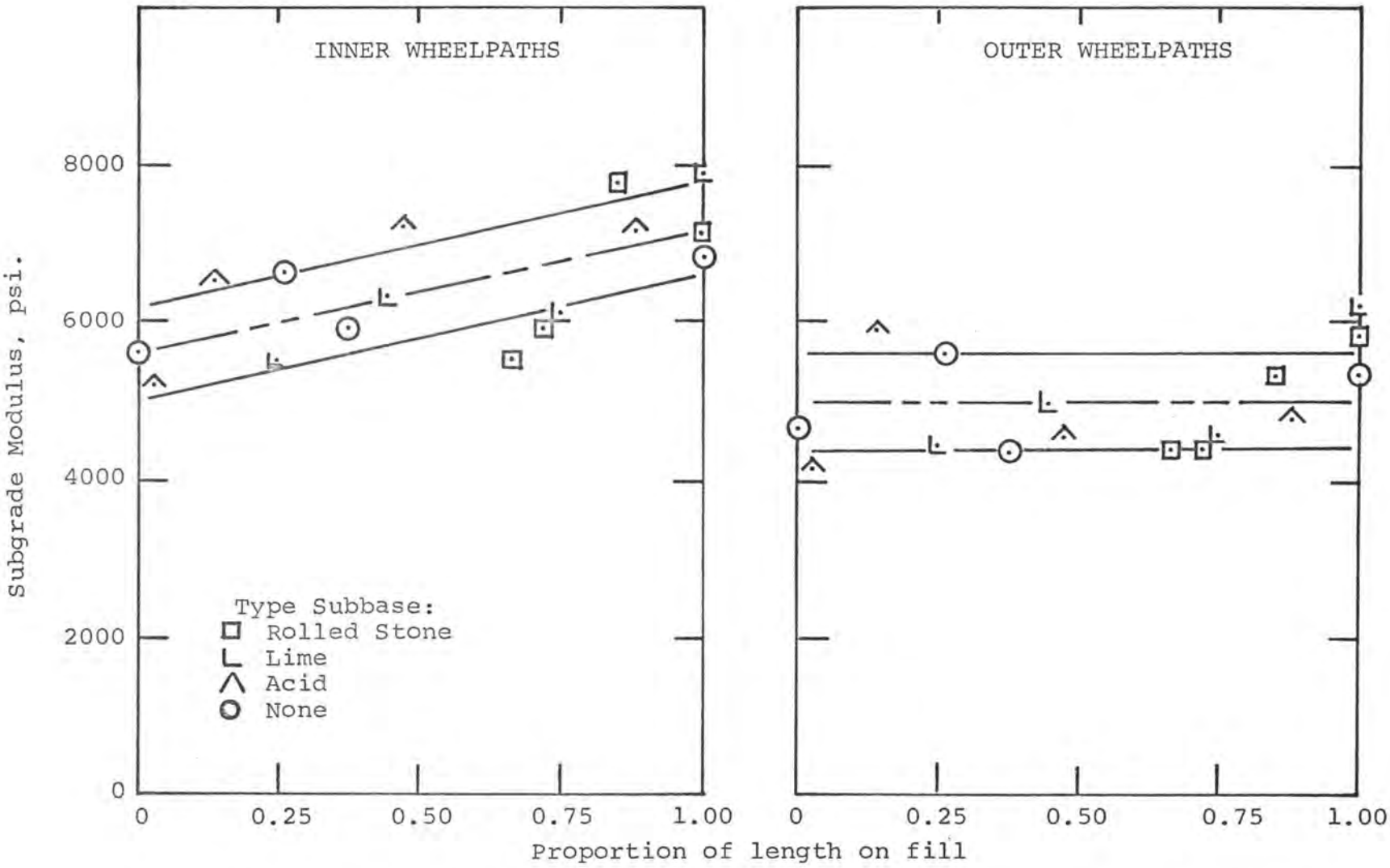


FIGURE 10 - COMPUTED SUBGRADE MODULUS IN EACH WHEELPATH VERSUS PERCENT LENGTH ON FILL FOR EACH TEST SECTION.

would not be significantly different from one with zero slope. Accordingly, the line is drawn at the average value of 5000 psi. for all test sections. The standard deviation is 600 psi. for each kind of wheelpath and the limiting lines are for plus or minus one standard deviation. The slopes of the regression lines in Figure 10 indicate that the assumption of a fairly uniform subgrade modulus for all test sections is (a) not false for the outer wheelpaths, and is (b) not true for the inner wheelpaths. It was noted for the analysis of variance in Table 10 that the interaction (RH_e) was highly significant for the inner wheelpaths. Although other conditions can produce a significant interaction, the non-uniformity of the test-section subgrade modulus in the inner wheelpath is no doubt responsible in this case. Figure 10 also implies that, (c) where lateral support of the subgrade is not a variable, as in the inner wheelpaths, measured values of the subgrade modulus would indicate a real difference between cut and fill, and (d) where lateral support is an additional variable, as in the outer wheelpaths, measured values of the subgrade modulus would not indicate any real difference between cut and fill.

EQUIVALENT THICKNESS REQUIRED

By inference from equation (5) the equivalent thickness required (H'_e), to restrict 50 percent of the Benkelman beam deflections under the axle load (L) to less than a tolerable deflection (U'), can be computed from the relation:

$$H'_e = L/2\ U'K_a \tag{10}$$

if K_a, the anticipated average value of normalized unit strength (K), is available from previous Benkelman beam measurements; or can be calculated by equation (5) using estimated values of the subgrade modulus and the relative stiffness factor.

In order to compute the equivalent thickness required to restrict more than 50 percent of the deflections to a tolerable value, the distribution of the individual values of K must also be considered. The following equation is pertinent:

$$H_e' = \frac{L}{2\ U'K_a\ (1 + t\ V'/100)} \tag{11}$$

where: t = the standard normal deviate, i.e., the difference, $K' - K_a$, divided by the standard deviation of the population.

$K' = K_a(1 + t\ V'/100)$ = a specific value of the unit strength.

V' = the coefficient of variation of the population of unit strengths from which samples are measured, it is equal to the estimated standard deviation of the population divided by the average unit strength, and expressed as a percentage.

A normal distribution is assumed for the population of unit strengths, and the probability (p_n) that some individual unit strengths are less than K' can be found in tables of the normal distribution (left tail). Since "p_n" is also the proportion of individual strengths expected to be less

than K', then $1-p_n$ is the proportion expected to be greater than K', and presumably $1-p_n$ is also the proportion of deflections restricted to the tolerable value U'. Some numerical values of p_n corresponding to values of t are:

t	p_n	$1-p_n$
0.00	.50	.50
-1.00	.16	.84
-1.28	.10	.90
-1.64	.05	.95

The State of California shows tolerable levels of deflection under a 15 kip axle load versus equivalent 5000 lb. wheel load (EWL) in Figure 6 of reference (8). ("By utilizing an average AC surfacing fatigue line slope (from laboratory tests) and pivoting lines through known deflection criteria at the 9.0 traffic index (TI) level, Figure 6 was developed for the purpose of making "rule of thumb" adjustment in tolerable deflection for varying traffic volumes".) The lines shown for tolerable deflection can be approximated by the relation:

$$\log MU' = 1.95 + \log L - \log (1.5 + h) - (\log EWL)/6 \quad (12)$$

By substitution of 16 for the ratio EWL/SL, relation (12) can be converted to an appropriate relation for equivalent 18 kip axle load:

$$\log MU' = 1.75 + \log L - \log (1.5 + h) - (\log SL)/6 \quad (13)$$

where:

MU' = tolerable deflection under load (.001 in.),

L = axle load, 15 kip for (12), and 18 kip for (13),

h = thickness of asphaltic concrete, in., and

SL = accumulated applications of equivalent 18 kip axle load.

The results of several computations, by relation (13), for tolerable deflection under an 18 kip axle load are shown in Table 13, as tentative tolerable deflections for some thicknesses of asphaltic concrete, and for various accumulated applications of equivalent 18 kip axle load. Values for tolerable deflection of a pavement with asphaltic concrete and black base are not shown in reference (8). The values shown in Table 13 for this combination may be applicable if the layers are bonded together and if the elastic moduli of the layers are not appreciably different.

Table 13

TENTATIVE TOLERABLE DEFLECTIONS (.001 IN.) UNDER AN 18 KIP AXLE LOAD

Thousand Applications[a] / Thickness	10	30	100	300	1000	3000
1 in. Asphaltic Concrete	87	73	60	50	41	34
2 in. Asphaltic Concrete	62	52	43	35	29	24
3 in. Asphaltic Concrete	49	40	33	27	22	19
4 in. Asphaltic Concrete	40	33	27	22	18	15
5 in. Asphaltic Concrete	34	28	23	19	16	13
6 in. Asphaltic Concrete	29	24	20	17	14	11
8 in. AC and Black Base	23	19	16	13	11	9
10 in. AC and Black Base	19	16	13	11	9	7
12 in. AC and Black Base	16	13	11	9	7	6

(a) Accumulated applications equivalent 18 kip axle load.

Samples of the traffic volume and axle load on Route 169, Gentry County indicated an annual southbound rate of 13 thousand equivalent 18 kip axle load applications (Table 11). The accumulated value for 10 years service would be about 130 thousand applications. With this figure and

Table 13 as a guide, a value of .032 in. is selected as a tolerable deflection for three inch thickness of asphaltic concrete. Before the equivalent thickness required can be computed by equation (11), a value must also be assigned to the coefficient of variation.

Table 10 indicates that the standard deviation of replicate test sections (error) is 6.3 in the inner wheelpaths, and 5.1 in the outer wheelpaths for 128 degrees of freedom. Each is equal to 19 percent of its average unit strength. The original data for Table 10 was average values of 7 measurements. Accordingly the coefficient of variation of individual unit strengths for items in statistical control might be computed as $19(7)^{\frac{1}{2}} = 50$ percent. This value appears to be too high. Examination of the subsets of seven deflections indicated the following coefficients of variation within replicates: 20 percent for 1962, 22 percent for 1963-64, and 30 percent for 1965-1966. Using these values as criteria, it can be shown that the replicates are not in statistical control, i.e., random variations between replicates due to random test section locations on the roadway are greater than predicted by the variation within test sections. (This has also been noted in preliminary examination of the data from one satellite test project.) Had Table 10 included an additional source for tests within replicates, with 1536 additional degrees of freedom, the mean squares shown would all become 7 times larger; and the effect of replicates (error in Table 10) would have been indicated as highly

significant, but its standard deviation would have been reduced slightly. The coefficient of variation for replicates would be about 17 or 18 instead of 19 percent. Because of the above, it is assumed that the two effects, between replicates and within replicates, are independent and random. Then the variance of the two effects may be added to obtain the variance of individual tests; or the squares of the two coefficients of variation may be added to obtain the square of the coefficient of variation V', since the average unit strength is the same for each effect. Selecting the values of 18 percent for between replicates and 22 percent for within replicates, the value for V' becomes 28 percent.

Substituting values of 18 for L, .032 for U', -1.00 for t, and 28 for V' into equation (11), the equivalent thickness required to restrict 84 percent of the deflections to a tolerable level is:

$$H_e' = \frac{281}{K_a\ (1 - 0.28)} = \frac{390}{K_a} \qquad (14)$$

This thickness is presumably the total effective thickness required to limit fatique cracking of a three inch thick asphaltic concrete surface to no more than 16 percent ($t = -1.0$, $p_n = 0.16$) of the pavement area after 130 thousand applications of equivalent 18 kip axle load. Of course, it is also assumed that the thickness is adequate to restrain shear displacement or plastic flow. If fatigue cracking due to flexing is to be limited to no more than five percent ($t = -1.64$, $p_n = 0.05$) of the pavement area, then the thickness required would be 33 percent greater than that shown by

equation (14), since $(1 - 0.28)/(1 - 1.64 \times 0.28) = 1.33$. Solutions of equation (14) are shown in Table 14 for the years 1962 to 1966. A study of the results indicates that:

1. The required equivalent thickness is greater for the outer wheelpaths than it is for the inner wheelpaths.
2. The equivalent thickness as built was inadequate in the outer wheelpaths for all test sections after two years service.
3. Additional study of the variations within and between replicates on other projects is essential to establish likely values for the coefficient of variation V'.

PERFORMANCE

The flexible pavement test sections were rated by a present serviceability index, (PSI) made in 1966. Our CHLOE profilometer was acquired in 1965, hence, trend values for PSI are not available. The results of the 1966 survey are shown in Table 15. Considering a PSI rating of 2.50 as a terminal value, then in October of 1966 after some four years of service at least one of four test sections for each type of subbase, rolled stone excluded, would be considered a failure.

An estimated performance index, est. P, for each test section can be obtained from equation, D10, shown in the Guidelines, reference (9), as:

$$\text{est. } P = \log SL + B\,(\log\log(p_o/2.5) - \log\log(p_o/p_i)) \qquad (15)$$

Table 14

EQUIVALENT THICKNESS REQUIRED IN EACH WHEELPATH

Year	1962	1963	1964	1965	1966
Month	Oct.	April	April	April	May
Normalized-unit-strength average, K_a, kip per in. deflection per in. thickness					
Inner Wheelpaths	43	33	37	29	34
Outer Wheelpaths	38	31	29	19	23
Equivalent Thickness Required to restrict 84 percent of the deflections to no more than 0.032 in. under an 18 kip axle load. $H'_e = 390/K_a$, in.					
Inner Wheelpaths	9	12	11	13	11
Outer Wheelpaths	10	13	13	20	17

The equivalent thickness of the test sections as built is shown below for comparison.

Design Thickness, in.	3-8-5	3-4-5	3-8-0	3-4-0
Type Subbase				
Rolled Stone	16	12		
Hydrated Lime	13	9[a]		
Phosphoric Acid	12[a]	8		
None	--	--	11	7[a]

(a) Table 15 shows 19 percent or more major cracking and patched area for at least one test section at age four years.

TABLE 15 - VALUES FROM CONDITION SURVEY OF TEST SECTIONS

		May 1966				October 1966			
Design	T. Sec.	PSI[a]	C+PA[b]	RD[c]	U[d]	PSI[a]	C+PA[b]	RD[c]	U[d]
Rolled Stone Subbase:									
3-8-5	4	3.87	5	.21	.031	3.65	2	.23	.027
3-8-5	6	3.87	0	.18	.025	3.74	0	.20	.028
3-4-5	10	3.91	1	.20	.027	3.82	1	.20	.031
3-4-5	16	3.76	2	.21	.038	3.52	2	.21	.043
	Average	3.85	2	.20	.030	3.68	1	.21	.032
Lime Subbase:									
3-8-5	1	3.34	32	.27	.037	3.37	27	.29	.039
3-8-5	3	3.36	7	.26	.030	3.42	9	.29	.030
3-4-5	9	3.81	7	.20	.033	3.79	13	.24	.039
3-4-5	11	3.18	103	.31	.050	2.40	306	.28	.046
	Average	3.42	37	.26	.038	3.25	89	.28	.038
Acid Subbase:									
3-8-5	2	2.60	189	.33	.046	2.25	189	.30	.038
3-8-5	8	3.78	2	.16	.024	3.67	3	.20	.027
3-4-5	14	3.91	5	.24	.045	3.36	3	.25	.060
3-4-5	15	3.49	36	.25	.049	3.40	48	.27	.053
	Average	3.44	58	.25	.041	3.17	61	.26	.044
No Subbase:									
3-8-0	5	3.58	11	.19	.047	3.45	13	.26	.048
3-8-0	7	3.76	2	.19	.028	3.65	4	.22	.033
3-4-0	12	3.03	55	.25	.062	2.57	298	.18	.052
3-4-0	13	2.02	660	.33	.070	1.63	722	.13	.060
	Average	3.10	182	.24	.052	2.82	259	.20	.048

(a) Present serviceability index.
(b) Major cracking and patched area, sq. ft. per 1000 sq. ft.
(c) Outer wheelpath average rut depth, in.
(d) Outer wheelpath average Benkelman beam deflection, in.

Let: est. P = log SY + log (365 ADL), and

log SL = log Y + log (365 ADL).

Substituting these expressions for est. P and log SL in relation (15) and then subtracting log (365 ADL) from both sides of the result, an estimate of the expected service life of each test section is obtained as:

$$\log SY = \log Y + B\ (\log \log (p_o/2.5) - \log \log (p_o/p_i)), \quad (16)$$

where:

est. P = the logarithm of the estimated accumulated applications of equivalent 18 kip axle load that should produce a terminal serviceability index of 2.50,

SL = the accumulated applications of equivalent 18 kip axle load when the present serviceability index is p_i,

p_o = the original serviceability index, assumed as 4.20 herein,

ADL = average daily applications of equivalent 18 kip axle load,

365 ADL = average annual applications of equivalent 18 kip axle load,

SY = expected service life of a test section, years,

Y = years service when the present serviceability index is p_i, and

B = the slope for relations (15) and (16).

When only one pair of values for Y, and p_i is available, it is necessary to assume a numerical value for the slope, B; however, the Guidelines state on page D6 "Experience with

the AASHO Road Test performance data has indicated that suitable approximations for B are,

B = 1.0 for performance records that are concave upward,

B = 0.5 for performance records that are concave downward."

Estimates for the expected service life of each test section are shown in Table 16, and were made by solving relation (16), with assumed slopes of, B = 1.00, B = 0.75, and B = 0.50. Values of p_i (PSI) used were those shown in Table 15 for October, 1966, at the age of four years. When this project was five years old it was scheduled to be resurfaced within the next 5 years, therefore, the service lives estimated for B = 1.00 seem too great. The service lives estimated for the other two slopes appear to be reasonable, but without additional data it cannot be said that one of these is better than the other. The point of interest is that the present serviceability index, or the estimate of service life, has ranked the four types of subbase in the same order as that previously obtained by evaluation of the deflection data. The expected service lives show positive correlations with the other varibles tabulated in Table 16. The correlation coefficients, r, for regressions of the logarithm of these variables on the logarithm of the service life are: (a) r = 0.56 for equivalent thickness of pavement H_e, (b) r = 0.63 for computed values of the subgrade modulus E, (c) r = 0.80 for percent length of test section on fill, and (d) r = 0.86 for the ratio of load to deflection L/2U; and each is significant for 16 points. The values tabulated in Table 16

Table 16

EXPECTED SERVICE LIVES OF TEST SECTIONS

Test Sec.	Design in.	H_e in.	E, psi.	% length on fill	L/2U kip/in.	Service Life, Years B=1.00	B=.75	B=.50
Rolled Stone Subbase:								
4	3-8-5	16	4300	67	360	14.7	10.7	7.4
6	3-8-5	16	5600	87	460	17.8	12.3	8.3
10	3-4-5	12	6300	100	390	21.8	14.5	9.3
16	3-4-5	12	4700	73	300	11.8	8.9	6.8
Average		14	5200	82	380	16.5	11.6	8.0
Lime Treated Subbase:								
1	3-8-5	13	4300	75	290	9.3	7.6	6.2
3	3-8-5	13	5000	44	340	10.0	8.0	6.3
9	3-4-5	9	6900	100	320	20.0	13.5	8.9
11	3-4-5	9	4500	25	210	3.7	3.8	3.8
Average		11	5200	61	290	10.7	8.2	6.3
Acid Treated Subbase:								
2	3-8-5	12	4000	3	250	3.3	3.5	3.6
8	3-8-5	12	6200	13	390	15.6	11.0	7.8
14	3-4-5	8	5600	48	240	9.3	7.6	6.0
15	3-4-5	8	5900	87	250	9.8	7.8	6.3
Average		10	5400	38	280	9.5	7.5	5.9
5	3-8-0	11	4200	37	240	10.5	8.3	6.5
7	3-8-0	11	6500	100	370	14.8	10.7	7.6
12	3-4-0	7	5200	26	180	4.2	4.2	4.1
13	3-4-0	7	4300	0	160	2.2	2.5	3.0
Average		9	5000	41	240	7.9	6.4	5.3

for L/2U and E, are averages of eight values for the year 1966 (two seasons, two wheelpaths, and two lanes).

Figure 11 shows a logarithmic plot of average deflection versus estimated years service from Table 16, for the slope B = 0.50. The deflection points plotted for each test section are averages of 16 values (four years, two seasons, and two lanes). The limiting lines are for one standard deviation of the logarithm of the deflection, or of the estimated service life. The transformed line of regression in terms of the estimated performance index, est. P, for the southbound traffic lane, and for the inner wheelpaths is:

$$\text{est. } P = 6.90 - 1.43 \log MU \tag{17}$$

and for the outer wheelpaths is:

$$\text{est. } P = 7.38 - 1.62 \log MU \tag{18}$$

where MU = deflection under an 18 kip axle load (.001 in.). The northbound traffic lane performance indexes would be 0.10 smaller. The constants in equations (17) and (18) will vary somewhat with the value assumed for the slope B. When trends of serviceability loss are available, (two or more measured values of PSI well separated) then values of the real slope can be computed. We are now accumulating data from a number of satellite test sections on other projects which should provide more reliable values for the constants. Then it will be possible to compare the results of Missouri tests with those for other regions, e.g., in reference (10) F. H. Scrivner and W. M. Moore report a critical analysis of Serviceability Loss as a Function of

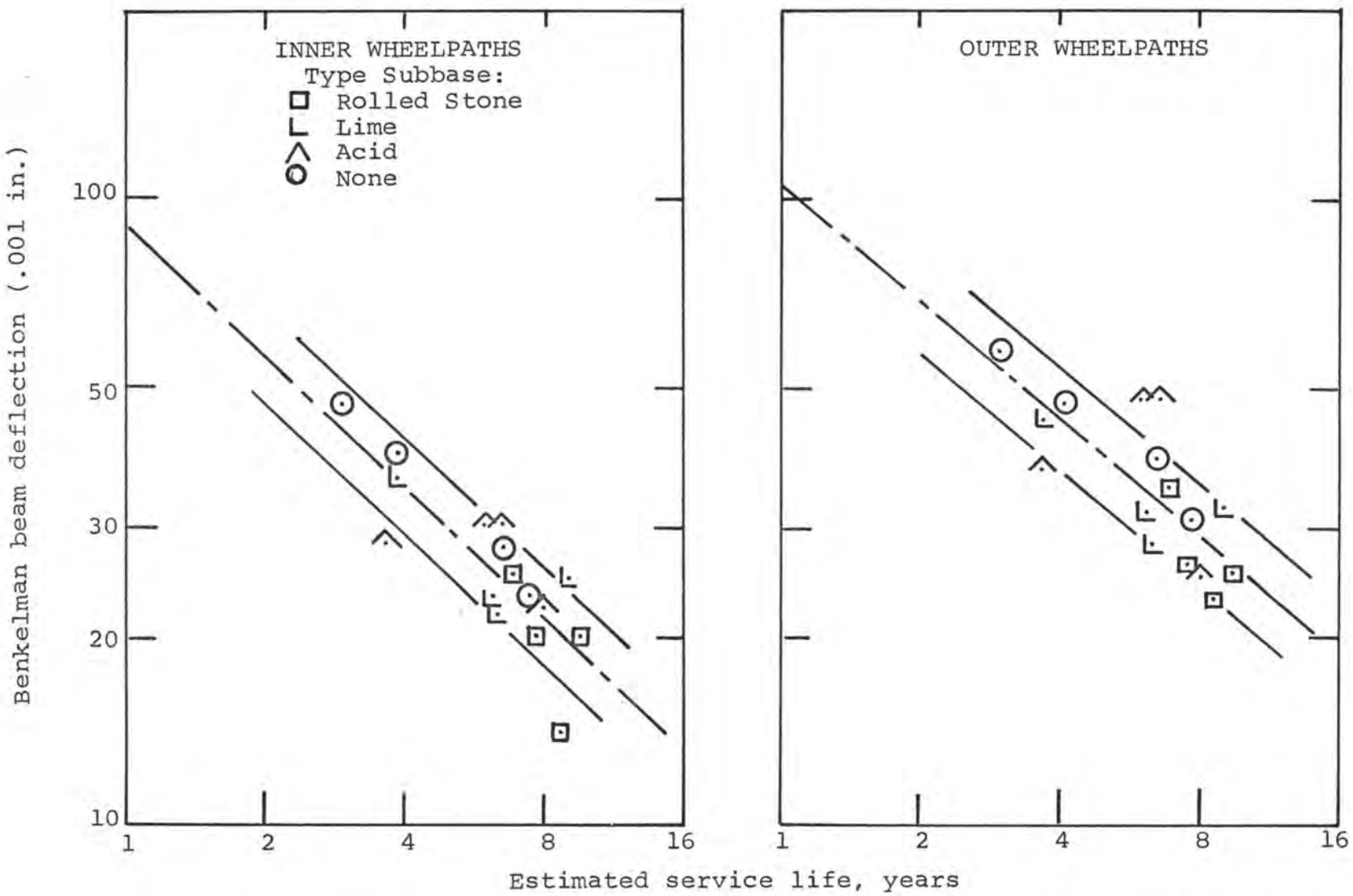


FIGURE 11 - AVERAGE DEFLECTION UNDER 18 KIP AXLE LOAD IN EACH WHEELPATH FOR SPRING AND FALL SEASONS, 1963 TO 1966, INCLUSIVE, VERSUS ESTIMATED YEARS SERVICE TO TERMINAL SERVICEABILITY INDEX OF 2.50.

Deflection for 21 flexible test sections with total thickness of 14 to 31 in. that survived the applications of single axle loads of 12 to 30 kip at the AASHO Road Test. For application to Texas test sections, they state:

"$\log W - \log \log (p_o/p_i) = 9 - 1.5 \log U$ (6A)

where W = the equivalent number of 18-kip single axle loads applied to a section as its serviceability index drops from p_o to p_i, and U = the deflection produced by a 9000 pound wheel load (in thousandths of an inch), assumed constant throughout the life of the section. The assumption that U is constant throughout the life of a Texas section may be questionable. However, the severe freeze-thaw cycles that occurred at the Road Test obviously were largely responsible for the wide fluctuations in deflection trends observed there. Since such cycles are rare indeed in Texas, we feel that seasonal variations are relatively minor in this state*.

---

* It is planned to check this assumption in future research."

If $p_o = 4.2$, and $p_i = 2.5$, and MU = U (to conform with notation herein), then an estimate of the performance index can be obtained from the above equation as:

$$\text{est. } P = 8.35 - 1.5 \log MU \qquad (19)$$

which is for the assumption of a constant deflection throughout the service life, whereas equations (17) and (18) are based upon an average of measured spring and fall deflections for the first four years, and a one point evaluation of the serviceability trend.

FINIS

This report is the final report for the Gentry County project.

REFERENCES

(1) Lime and Phosphoric Acid Stabilization in Missouri, C. E. Thomas, W. G. Jones, and W. C. Davis, Highway Research Record Number 92, 1965.

(2) Principles of Pavement Design, by E. J. Yoder, John Wiley & Sons Inc., 1959.

(3) C. A. Hogentogler, Jr. page 329, Vol. 20, Highway Research Board Proceedings, 1943. Mr. Hogentogler was discussing an article by L. A. Palmer and E. S. Barber entitled "Soil Displacement Under a circular Loaded Area", page 279, Vol. 20, HRB Proceedings, 1943.

(4) Statistical Analysis in Chemistry and the Chemical Industry. Carl A Bennett and Norman L. Franklin. John Wiley & Sons, Inc., 1954.

(5) Statistical Theory and Methodology in Science and Engineering by K. A. Brownlee, John Wiley & Sons, Inc., 1960.

(6) NCHRP Report 10, Theroretical Analysis of Structural Behavior of Road Test Flexible Pavements, by Aleksandar Sedmak Vesic and Leonard Domaschuk, Georgia Institute of Technology, Atlanta, Georgia, 1964.

(7) NCHRP Report 35, Prediction of Flexible Pavement Deflections From Laboratory Repeated - Load Tests, by H. B. See, F. G. Mitry, C. L. Monismith, and C. K. Chan, Institute of Traffic and Transportation Engineering, University of California, 1967.

(8) Pavement Deflection Research and Operations since 1938. State of California, Transportation Agency, Department of Public Works, Division of Highways, Materials and Research Department, April 1966.

(9) NCHRP Report 2A, Guidelines for Satellite Studies of Pavement Performance, 1964.

(10) A Tentative Flexible Pavement Design Formula and Its Research Background, by F. H. Scrivner and W. M. Moore, Research Report Number 32-7, Texas Transportation Institute, College Station, Texas, 1966.

MoDOT Library
RD0011421