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Design and Testing of Flexible Pavement

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Design and Testing of Flexible Pavement

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Contents

Wheel-Load-Stress Computations Related to Flexible Pavement Design

CHESTER McDOWELL, Senior Soils Engineer,
Texas Highway Department

● IT is recognized that no exact method for calculation of stresses beneath a wheel load exists; however, it is generally conceded that stresses existing below a uniformly loaded circular area are approximately the same as those existing beneath a pneumatic tire of similar contact area and pressure intensity. It has also been conceded by many that A. E. H. Love's solution of Boussinesq's equations of elasticity presents a desirable method of estimating stresses beneath a wheel load.

The Texas triaxial method of flexible pavement design was influenced a great deal by calculations involving the above mentioned theories. Although the Texas method has been developed largely for testing of materials taken from roads of known behavior, the question is often asked about what stress considerations are pertinent to the method in use. In an attempt to answer this question the writer has tried to present some specific calculations, based on certain assumptions, which will afford a comparison between their results and those of the Texas Highway Department method. This report represents only one approach to calculation of wheel load stresses out of many possible ones; therefore, it would not be unusual for the reader to prefer other assumptions and solutions to the problem.

The assumptions used are as follows:

1. The theory of elasticity involving:
   (a) A. E. H. Love's solution for stresses beneath a circular loaded area.
   (b) Warner H. Tuft's computation of influence values for Love's equations.
   (c) That the use of tangential stress omega-omega, $\widehat{ww}$, in a few cases where it is less than $\frac{1}{2}$ ($\widehat{zz}$ + $\widehat{pp}$ - s) would not alter results of calculations for this problem sufficiently to justify their inclusion in this report.
   (d) Reduction of shearing stresses obtained from Love's solutions due to layers of pavements having higher moduli than subgrades. This is accomplished by multiplying Love's maximum shearing stresses by a factor F_S. F_S is the ratio of the maximum shear on the axis of the loaded area from F. H. Scrivner's solution[1] for "layer two" of the two layered system to the maximum shearing stress on the axis of loaded area from Love's solution. Mr. L. E. McCarty of the Texas Highway Department developed F_S as shown in Table 2 and Figure 4-A. It may be noted that F_S is a function of depth Z/a and ratio of moduli E_2/E_1. A set of empirical values of E_2/E_1 based upon experience were selected by the author and are shown in Table 3.
   (e) The addition of effects of surcharge weight equal to 1 psi. per foot of depth upon normal stresses for depths greater than 12 inches.
2. Poisson's Ratio $\mu = 0.5$
3. A unit load of 100 psi. With exception of pavements located in zones where traffic makes extensive use of brakes for stopping, this unit pressure is thought to be high enough so as to eliminate the necessity of considering effects of added stresses due to acceleration or deceleration.
4. Wheel loads of 24,000 and 10,000 pounds were used in computations. Radii of loaded areas were 8.75 and 5.6 inches, respectively.

Calculation data are shown as follows:

1. Figure 1 shows geometrical dimensions used by Love in solution of the stress problem. Notations for Table 1 are also shown on Figure 1.
2. Table 1 lists Warner H. Tuft's computation of influence values for Love's equations.
3. Figures 2, 3, and 4 show the distribution of vertical stresses, radial or horizontal stresses and stress differences under a uniformly loaded circular area. The data

[1] Some Numerical Solutions of Stresses in Two and Three Layered Systems by F. H. Scrivner, Volume 28, Proceedings of the Highway Research Board.

SOME GEOMETRICAL DIMENSIONS USED BY LOVE IN THE CALCULATION OF STRESSES BENEATH A UNIFORMLY LOADED CIRCULAR AREA

NOTATIONS

a = Radius of loaded area

ρ = Radial distance in cylindrical co-ordinates

z = Depth below surface

ρ/a = Radial distance expressed in terms of radius

z/a = Depth expressed in terms of radius

p = Applied uniform stress

$\widehat{zz}$ = Stress component acting in a vertical direction upon a horizontal plane

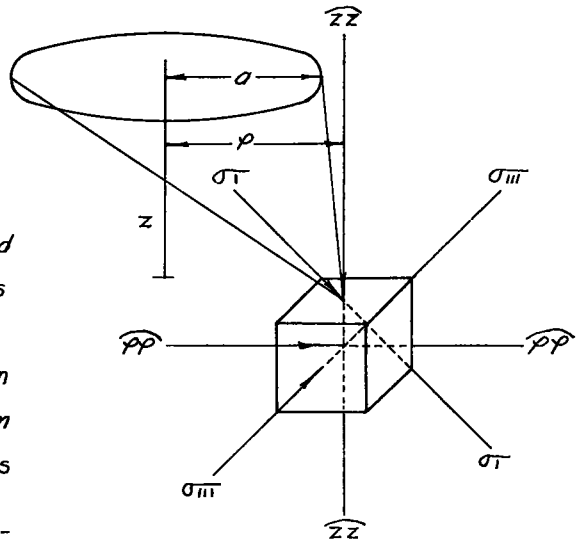
$\widehat{\rho\rho}$ = Stress component acting in a radial direction upon a plane perpindicular to the radius

S = Stress difference $= \sigma_I - \sigma_{III}$

μ = Poisson's ratio

$\sigma_I = \frac{1}{2}(\widehat{\rho\rho} + \widehat{zz} + S)$ Major principal stress

$\sigma_{III} = \frac{1}{2}(\widehat{\rho\rho} + \widehat{zz} - S)$ Minor principal stress*



*Tangential stress $\widehat{\omega\omega}$ not used. See text

Figure 1.

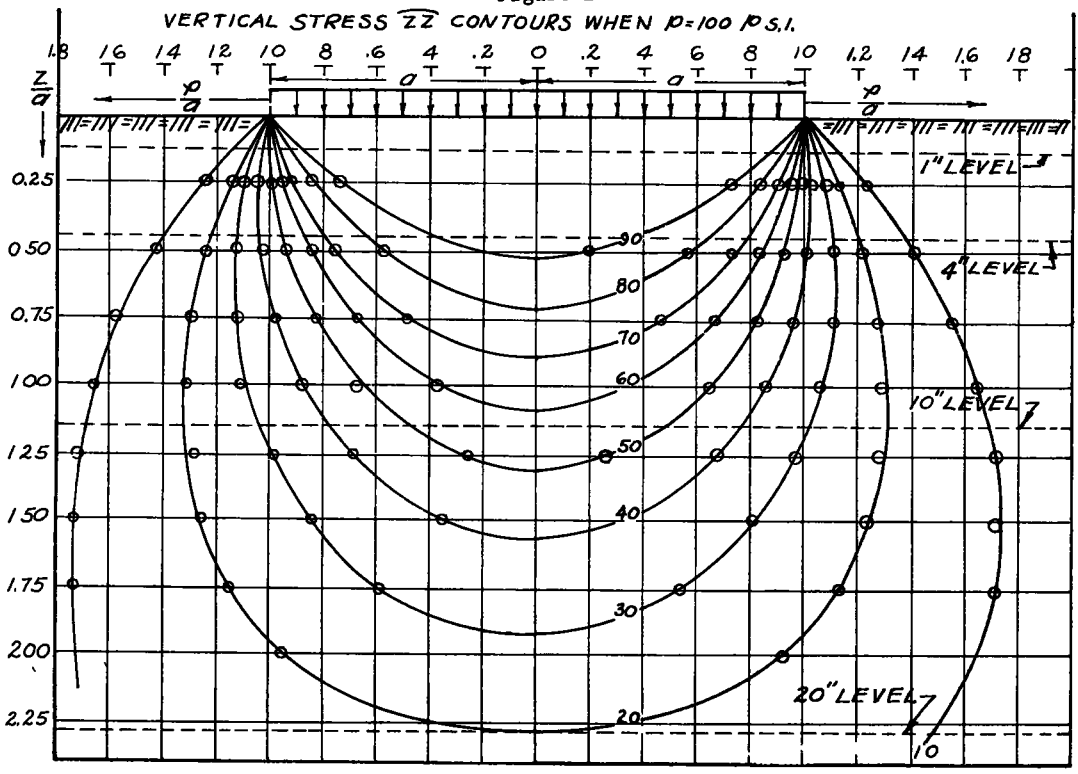


Figure 2.

TABLE 1

WARNER H TUFT'S COMPUTATION OF INFLUENCE VALUES FOR A E. H LOVE'S SOLUTION OF STRESSES EXISTING IN A SEMI-INFINITE ISOTROPIC SOLID (HAVING A POISSON RATIO OF 0.5) SUPPORTING A LOAD APPLIED UNIFORMLY OVER A CIRCULAR AREA

(Taken from 'Public Aids to Transportation Vol. IV")

$\frac{\rho}{a}$	$\frac{\widehat{\rho\rho}}{\rho}$	$\frac{\widehat{zz}}{\rho}$	$\frac{S}{\rho}$
	Z/a = 0.25		
0	6433	.9857	.3424
.0670	.6428	.9855	3429
.3131	6177	.9801	.3672
5670	5471	.9542	.4361
75	4580	8838	5322
8834	3970	.7243	.6080
1	.3849	.4596	6023
1.1166	3613	.2086	.4870
1.25	.2812	0737	3322
1.4330	1800	0211	.1961
1.6869	1015	0056	.1061
2 4178	0301	0005	0306
	Z/a = 0 5		
0	.3739	.9106	.5367
.1340	3703	.9066	5395
2859	3567	.8913	.5504
5	3249	.8396	.5754
.7668	2875	.6775	5968
1	2864	4175	5403
1 1820	2758	.2224	.4318
1 5	1962	.0604	.2448
1.7141	1430	0268	1657
2.0722	0848	0085	0924
2.8660	.0324	.0013	.0337
	Z/a = 0 75		
0	.2080	7840	5760
.1062	2079	.7803	5753
.3707	1999	.7354	5749
.5670	.1951	.6621	5662
.7990	.1989	.5241	.5326
1	2087	.3745	.4722
1.2010	.2094	.2353	3889
1 4330	.1886	1241	.2897
1 75	1419	0872	.1698
2.0711	.1031	.0639	.0977
2.6084	.0561	.0367	.0418
3.7990	.0196	0130	.0105
	Z/a = 1 00		
0	.1161	6464	.5303
1609	1174	6378	.5273
4226	1205	5864	5136
6360	.1287	5103	.4880
9125	.1453	3773	4305
1.1763	.1563	2461	3306
1.4663	.1499	1363	2654
1 8391	1205	0595	1734
2 4281	0732	0173	0893
3 1445	0397	0050	.0444
4 7321	.0131	.0006	0137

$\frac{\rho}{a}$	$\frac{\widehat{\rho\rho}}{\rho}$	$\frac{\widehat{zz}}{\rho}$	$\frac{S}{\rho}$
	Z/a = 1 25		
0	0668	.5239	.4571
.1247	0686	.5197	4545
.4171	0752	.4774	.4388
.6651	0878	4100	.4094
.8906	.1017	.3322	.3706
1 1094	.1130	.2534	3245
1.4550	.1179	1479	.2466
1 8753	1027	.0696	.1656
2.4897	0691	.0233	.0909
3.1651	.0422	0080	0499
4.4343	.0181	.0017	.0196
	Z/a = 1.5		
0	.0399	4240	.3840
.1340	.0417	.4203	.3815
.4540	0499	.3833	.3653
7355	.0637	.3240	.3357
1	0777	2562	2976
1.2645	0879	.1892	2540
1 5460	.0914	.1292	.2073
1.8660	.0866	0800	1567
2.2587	0736	0435	1143
2 7876	.0545	0197	.0732
3.5981	.0329	.0066	.0393
4.2168	.0227	0032	0195
	Z/a = 1 75		
0	.0249	.3455	3206
1840	0266	.3406	3179
5311	0361	.3058	3006
8469	.0496	.2532	2719
1 1531	.0623	.1947	.2362
1.4689	0704	.1386	.1963
1.8160	0717	.0901	1549
2 2254	0652	0523	.1143
2.75	.0522	.0259	.0769
3 4993	0351	.0102	.0449
4 7529	.0180	0027	.0206
	Z/a = 2.0		
0	.0161	2845	.2863
2721	.0189	.2767	.2640
6473	.0287	.2430	.2461
1.3527	.0519	.1459	1859
1.7279	.0575	.0996	.1502
2 1547	.0565	.0612	.1143
2.6782	.0488	.0329	.0802
3 3835	0361	0145	.0502
4.4641	.0215	0047	0261

$\frac{\rho}{a}$	$\frac{\widehat{\rho\rho}}{\rho}$	$\frac{\widehat{zz}}{\rho}$	$\frac{S}{\rho}$
	Z/a = 2.5		
0	.0075	.1996	.1921
.3301	.0098	1935	.1885
.7813	.0179	1681	.1742
1 2187	.0280	1327	.1524
1 6699	.0360	.0955	.1264
2 1658	.0394	.0621	.0988
2.7505	.0374	0357	0718
3 5	.0303	.0174	.0472
4.5703	.0204	.0066	.0271
	Z/a = 3.0		
0	0039	.1462	1423
.7375	.0098	.1300	.1327
1.2625	.0175	.1046	.1172
1 8038	.0245	.0761	0976
2 3989	0282	.0497	.0762
3 5173	0254	0203	.0452
4.5753	0190	.0088	0276
	Z/a = 4.00		
0	.0013	.0869	0856
.2947	.0023	0856	0840
1	.0056	.0761	.0789
1.7053	.0105	0564	0663
2.4559	.0148	.0463	0580
3 3094	0164	0355	.0452
4.3564	.0156	.0258	.0305
	Z/a = 5.00		
0	.0006	.0571	0566
1	.0026	.0523	.0534
1.8816	0060	0410	.0462
2 8198	0091	.0353	0417
4.5010	.0108	0241	0276
	Z/a = 7 00		
0	.0002	.0299	.0297
1	.0010	.0283	.0284
1 6124	.0017	.0245	.0257
2 8756	.0038	.0236	0258
3.5478	.0047	.0224	0246
4.2641	.0051	.0210	.0230

for plotting of these figures were taken from Table 1 assuming an applied unit load of 100 psi.

4. Figures 5, 6, 7 and 8 show plottings of Mohr diagrams of stresses for the 1, 4, 10 and 20 inch levels, respectively, beneath a circular loaded area having a unit pressure of 100 psi. and a radius 8.75 in. The stress value of $\widehat{zz}$ for vertical, $\widehat{\rho\rho}$ for radial, and $S/2$ for maximum shear which were used in plotting of the Mohr diagrams were interpolated from Figures 2, 3 and 4. These values are tabulated in the upper right part of each Mohr diagram chart.

5. Figure 9 shows the stress envelopes from Figures 5, 6, 7 and 8 superimposed on the Texas triaxial classification chart. The lowest[2] classes of soil materials strong enough to pass above the calculated stress envelopes are shown on left side of chart.

The foregoing method of presenting stresses, i. e., by the use of Figures 2, 3 and 4, is far from the easiest but is used in this instance to present a picture of stress distribution. For those who are accustomed to using tables of influence values and who are

[2] Lowest graphically meaning weakest class.

TABLE 2

SHOWING CALCULATION OF MAXIMUM SHEAR STRESS RATIOS USED IN FIGURE 4-A

Two Layered System

E_2/E_1	.01		.05		0.1		0.3		0.5		1.0		Isotropic Medium
Z/a	S/2	F_s	S/2	F_s	S/2	F_s	S/2	F_s	S/2	F_s	S/2	F_s	S/2
1.00	.0180	.068	.0595	.224	.0935	.353	.1701	.642	.2119	.798	.2650	.999	.2651
1.25	.0131	.057	.0453	.200	.0726	.317	.1394	.610	.1775	.777	.2285	1.000	.2285
2.00	.0062	.046	.0227	.169	.0379	.282	.0768	.573	.1005	.749	.1341	1.000	.1341
2.50	.0042	.044	.0156	.163	.0263	.274	.0541	.563	.0714	.744	.0960	1.000	.0960
5.00	.0011	.039	.0043	.152	.0074	.261	.0155	.548	.0207	.731	.0283	1.000	.0283

not concerned with values for an exact depth will find it easier to plot directly from tables of influence values.

6. Figures 10 through 14 show plotting of Mohr diagram of stresses where values for vertical, radial and maximum shearing stresses were taken directly from Table 1 for depths of 1.4, 2.8, 7, 14 and 28 inch levels beneath a circular loaded area having a unit load of 100 psi. and a radius of 5.6 inches. The depth levels represent Z/a ratios of 0.25, 0.50, 1.25, 2.50, and 5.00 respectively, and influence values for stresses at such depths can be taken directly from Table 1. These data are used for plotting of the

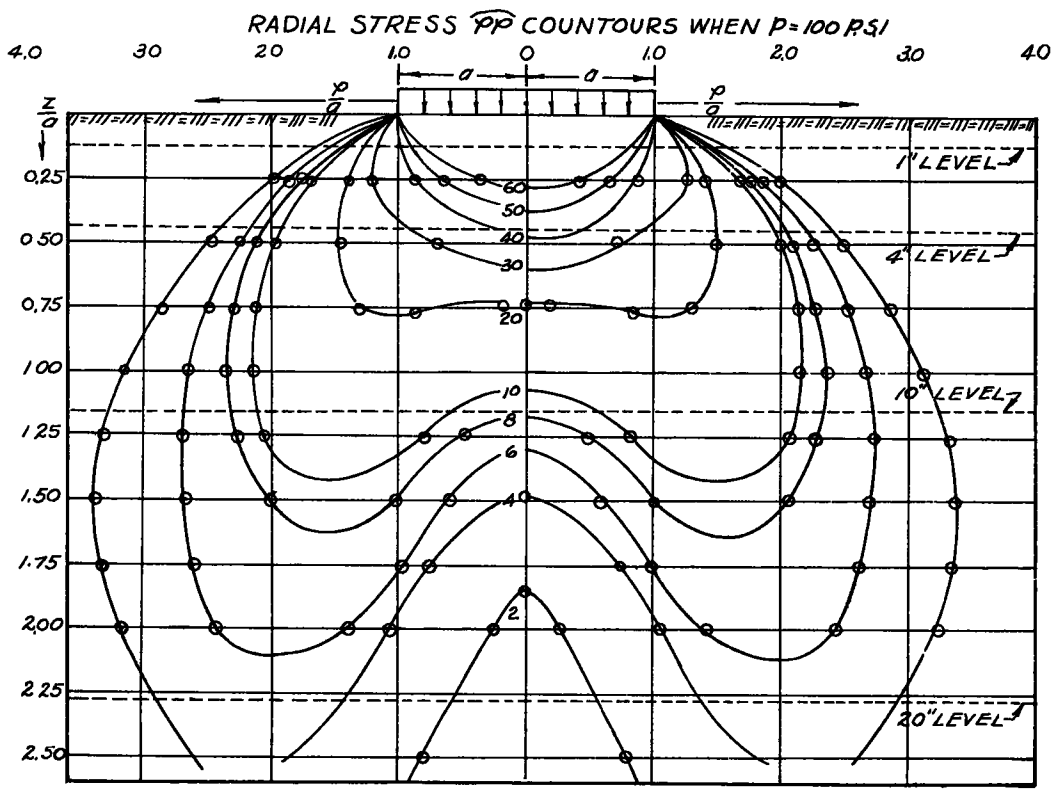


Figure 3.

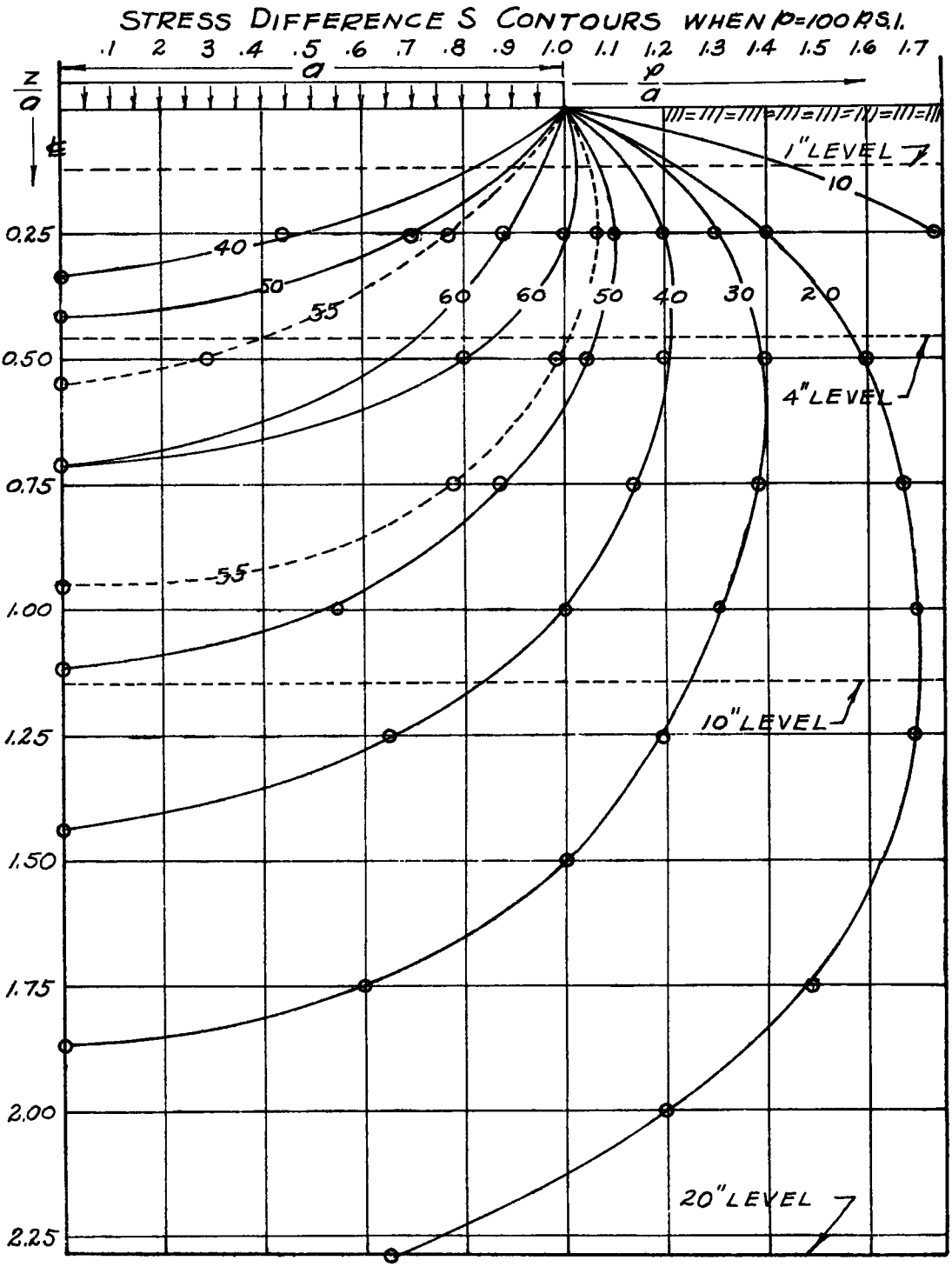

STRESS DIFFERENCE S CONTOURS WHEN P=100 P.S.I.
.1 .2 .3 .4 .5 .6 .7 .8 .9 1.0 1.1 1.2 1.3 1.4 1.5 1.6 1.7
a
p/a
z/a
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
1" LEVEL
4" LEVEL
10" LEVEL
20" LEVEL
10
40
50
55
60
60
50
40
30
20
55

Figure 4.

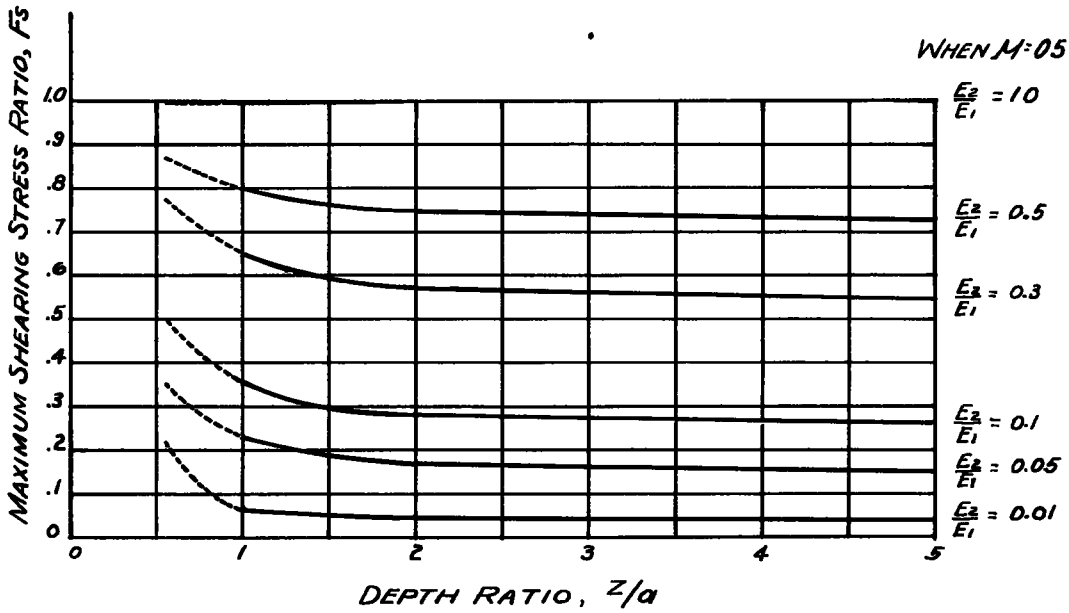


Figure 4-A. Shearing stress correction ratio, F_s, for converting Loves influence stresses to those for a two layered system.

ONE INCH LEVEL p = 100 psi., μ = 0.5, a = 8 75", F_s = 1 00

$\widehat{zz}$	ρ/a	$\widehat{\rho\rho}$	Z/a	Center $\widehat{zz}/2 + \widehat{\rho\rho}/2$	Radius S/2
96	℄	85	0.114	90	8
90	.88	50	"	70	25
80	.94	46	"	63	30
70	.96	44	"	57	31
60	.98	42	"	51	31
50	1.00	40	"	45	31
40	1 02	38	"	39	30
30	1.04	38	"	34	30
20	1 06	36	"	28	28
10	1 12	34	"	22	21

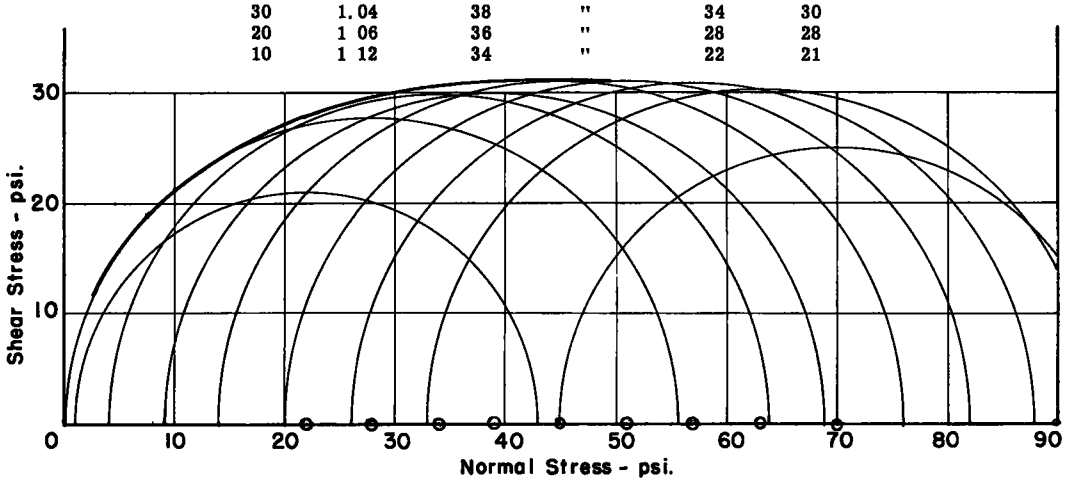


Figure 5.

FOUR INCH LEVEL, $\rho = 100$ psi., $\mu = 0.5$, $a = 8.75''$, $F_S = 0.96$

$\widehat{zz}$	ρ/a	$\widehat{\rho\rho}$	Z/a	Center $\frac{\widehat{zz}+\widehat{\rho\rho}}{2}$	Radius $S/2$	$S/2 \times 0.96$
94	℄	42	0 457	68	26	25
90	0.36	42	"	66	27.5	26 4
80	0 64	36	"	58	30	27 8
70	0.78	32	"	51	30	29.8
60	0.86	30	"	45	28.5	27 4
50	0.94	29	"	39 5	28	26.9
40	1.02	28	"	34	27	26
30	1.12	26	"	28	25	24
20	1.20	25	"	22.5	21	20.2
10	1.36	22	"	16	16	15 4

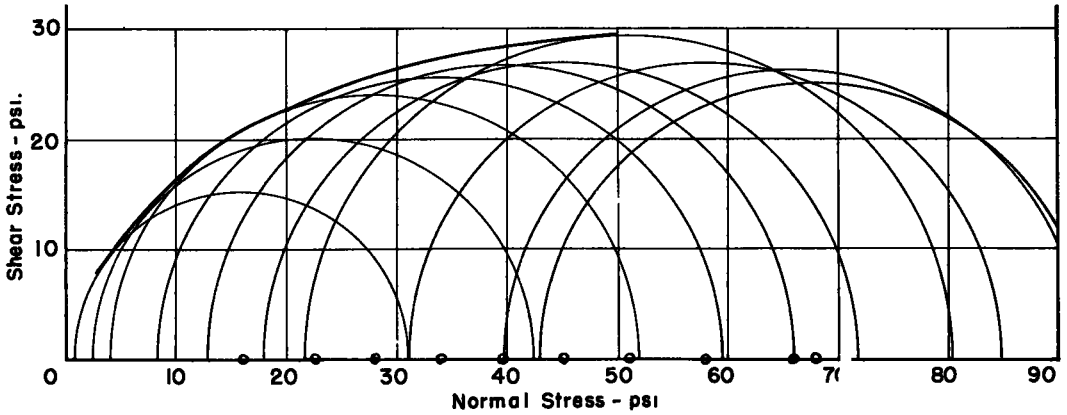


Figure 6.

TEN INCH LEVEL, $\rho = 100$ psi., $\mu = 0.5$, $a = 8.75''$, F_S · 1.91

$\widehat{zz}$	ρ/a	$\widehat{\rho\rho}$	Z/a	Center $\frac{\widehat{zz}+\widehat{\rho\rho}}{2}$	$S/2$	Radius $S/2 \times 0.91$
57	℄	9	1.142	33	25	22.7
50	0.43	10	"	30	23	20.9
40	0.76	11	"	25.5	21	19.1
30	1.00	12	"	21	18	16.4
20	1.28	12	"	16	15	13.6
10	1.68	12	"	11	11	10.0

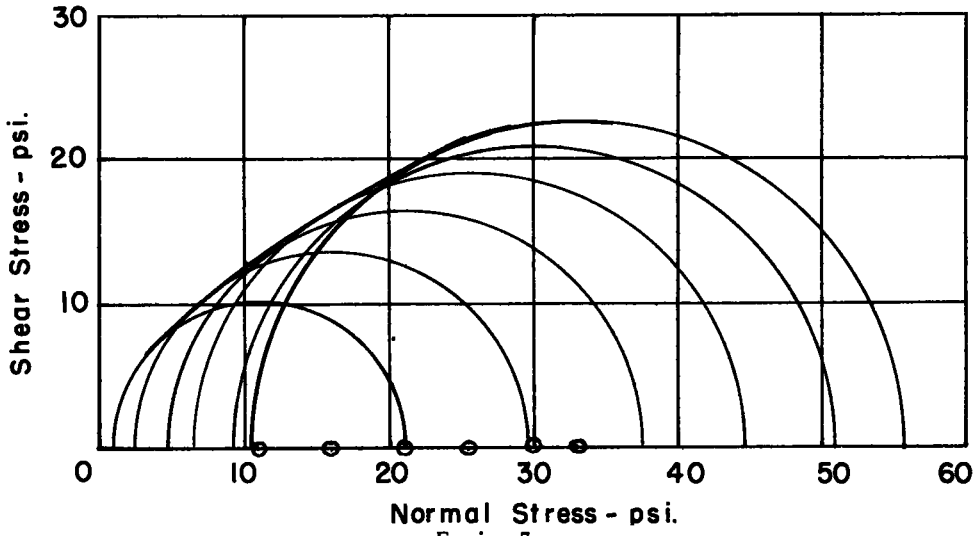


Figure 7.

TWENTY INCH LEVEL, ρ = 100 psi., μ = 0.5, a = 8.75", F_s = 0.82

$\widehat{zz}$	ρ/a	$\widehat{\rho\rho}$	Z/a	Center $\frac{\widehat{zz} + \widehat{\rho\rho} + 0.7^*}{2}$	S/2	Radius S/2x0.82
20	℄	2	2.28	11.7	10.5	8.6
15	1.115	3	"	9.7	8	6.6
10	1.500	4	"	7.7	7	5.8
5	2.350	5	"	5.7	5	4.1

* For all depths greater than 12 inches surcharge weight in psi. = $\frac{\text{depth level inches} - 12 \times 1}{12}$

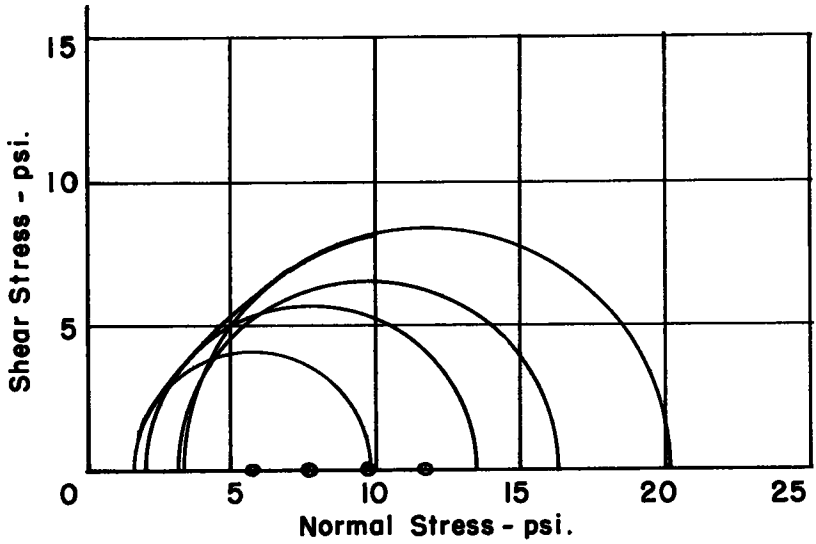


Figure 8.

TABLE 3

Depth Divided by Radius of Loaded Area Z/a	Approximate Average Value of Moduli[a] E_2/E_1	Shearing Stress Ratio Factor F_s
0.25	1.0	1.00
0.50	0.95	0.96
0.75	0.90	0.95
1.00	0.85	0.93
1.25	0.80	0.90
1.50	0.75	0.89
1.75	0.70	0.87
2.00	0.65	0.84
2.50	0.6	0.81
3.00	0.5	0.76
4.00	0.4	0.67
5.00	0.35	0.58

[a] These are empirical values based upon experience.

Mohr diagrams and are shown in the upper right hand portion of these charts.

7. Figure 15 shows stress envelopes from Figures 10 through 14 superimposed on the Texas classification chart. The lowest* class of soil strong enough to pass above calculated stress envelopes are also shown.

8. Table 4 shows depth levels at which stress computations were made, the weakest triaxial classifications not subject to overstressing, and the depths required for long-life roads built on such soil materials. These depths are taken from the Texas wheel load analysis chart, Figure 16. This chart has been worked out on the following basis:

(a) Depths are known for a given wheel load.

(b) Tire pressures remain constant when wheel loads vary and the magnitude

*Lowest graphically meaning weakest class.

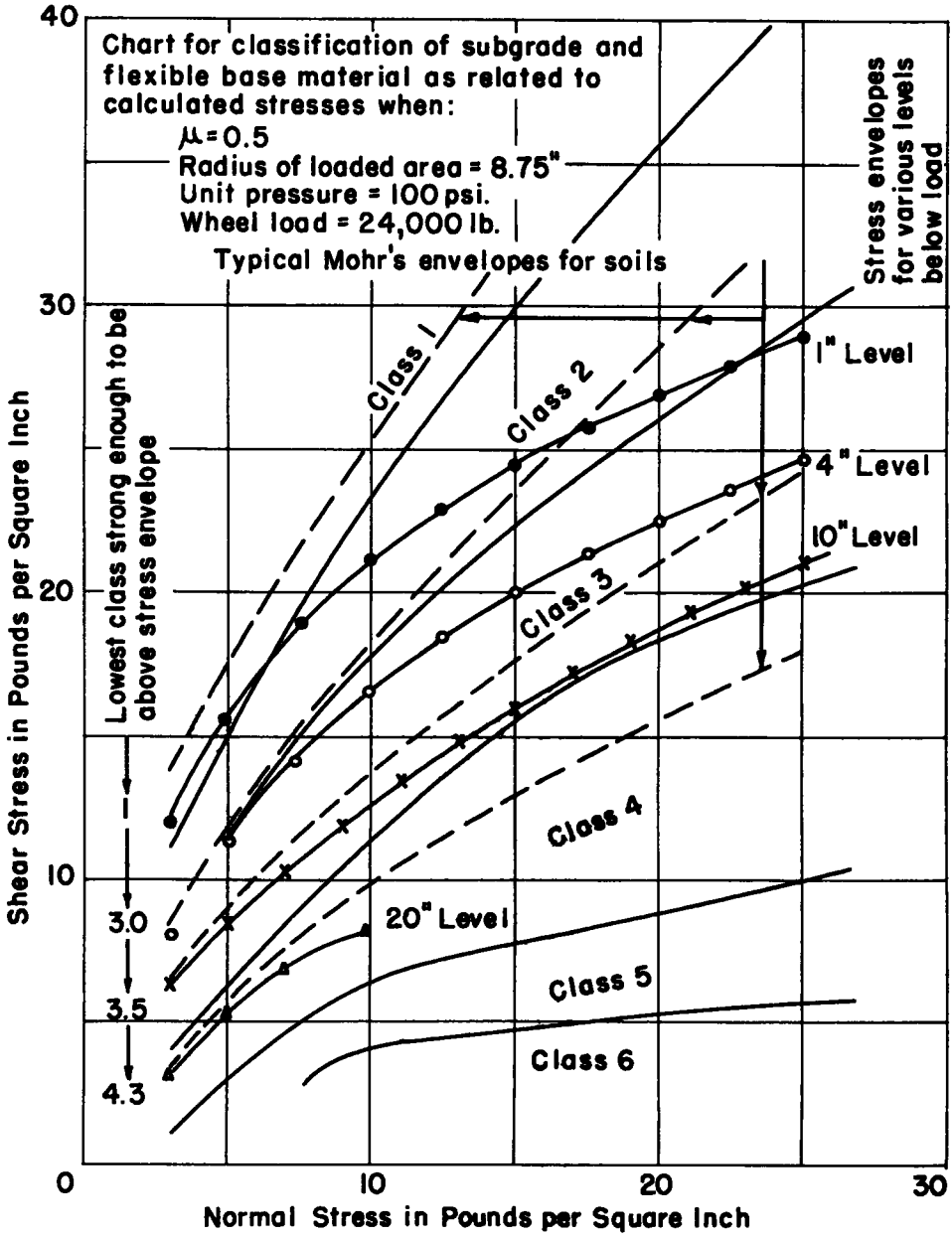


Figure 9.

of radii of contact areas will change accordingly.

Then: Depth for desired design wheel load = Known depth for given wheel load

$$\sqrt{\frac{\text{wheel load desired for design}}{\text{wheel load for which depths are known}}}$$

A comparison of levels shown in the last and third from last column of Table 4 indicates that the Texas triaxial method requires approximately the same depths as those required by the theoretical concepts presented herewith.

1.4 INCH LEVEL ρ = 100 psi., μ = 0.5, a = 5.6", F_s = 1.00

$\widehat{zz}$	ρ/a	$\widehat{\rho\rho}$	Z/a	Center $\frac{\widehat{zz}+\widehat{\rho\rho}}{2}$	Radius S/2
98.6	0	64.3	0.25	81.5	17.1
72.4	0.8834	39.7	"	56.1	30 4
46.0	1.000	38.5	"	42.3	30 1
20.9	1.1166	36.1	"	28.5	24.4
7.4	1.2500	28.1	"	17.8	16.6
2.1	1 4330	18.0	"	10.1	9.8

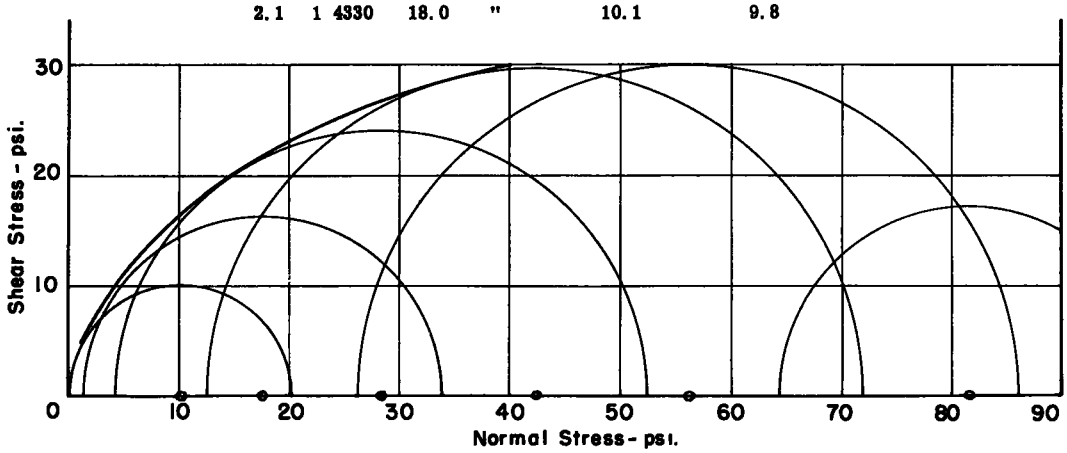


Figure 10.

2.8 INCH LEVEL ρ = 100 psi., μ = 0.5, a = 5.6", F_s = 0.96

$\widehat{zz}$	ρ/a	$\widehat{\rho\rho}$	Z/a	Center $\frac{\widehat{zz}+\widehat{\rho\rho}}{2}$	S/2	Radius S/2 x 0.96
91.0	0	37.4	0.5	64.2	26.9	25.8
84 0	0.500	32.5	"	58.3	28 8	27.6
41.8	1 000	28 6	"	35.2	27.0	25.9
22.2	1.1820	27.6	"	24.9	21 6	20 7
6.0	1.500	19.6	"	12.8	12.3	11.8
2 7	1.7141	14.3	"	8.5	8.3	8.0
0.8	2.0722	8.5	"	4.6	4.6	4.3

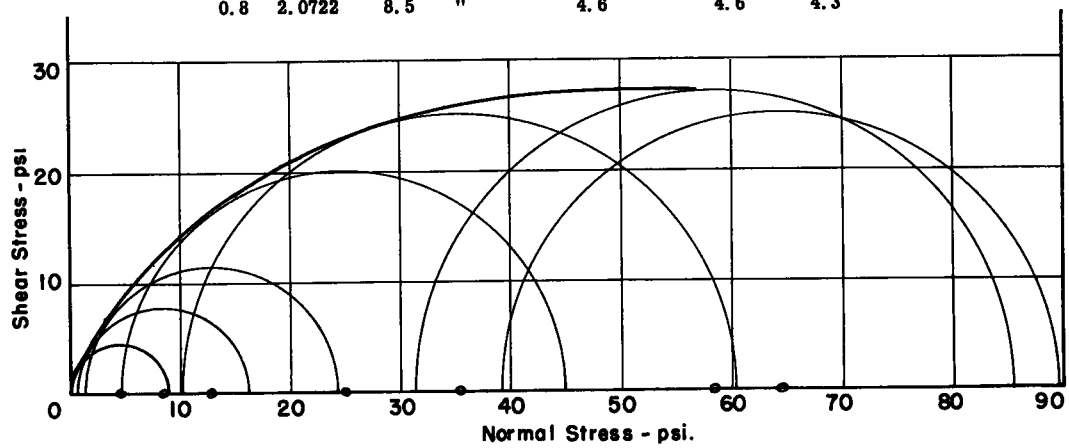


Figure 11.

TABLE 4

WHEEL LOAD DATA
UNIT PRESSURE = 100 PSI

Load in Lbs.	Radius (Inches)	Z/a Ratio of Depth to Radius	Depth Level at which Calculations were made (Inches)	Strength Class of Material Required to Prevent Overstress	Depth Required for Long-Life Roads (Inches)
24,000	8.75	0.114	1	1	1
"	"	0.457	4	3.0	5.3
"	"	1.14	10	3.3	9.0
"	"	2.28	20	4.3	20.5
10,000	5.6	0.25	1.4	2.7	2.5
"	"	0.50	2.8	3.3	5.5
"	"	1.25	7.0	3.7	9 0
"	"	2.50	14.0	4.0	11.5
"	"	5.0	28.0	6.4	28.0

7 0 INCH LEVEL, ρ = 100 psi , μ = 0.5, a = 5.6", F_s = 0.90

$\widehat{zz}$	ρ/a	$\widehat{\rho\rho}$	Z/a	Center $\frac{\widehat{zz}+\widehat{\rho\rho}}{2}$	S/2	Radius S/2 x 0.90
47.8	0.4171	7.5	1.25	27.7	22.0	19.8
41.0	0.6651	8.8	"	24.9	20.5	18.4
25.3	1.1094	11.3	"	18.3	16.2	14.6
14.7	1.4550	11.8	"	13.3	12 3	11 1
7.0	1.8753	10 3	"	8 6	8.3	7 5
2 3	2.4897	6.9	"	4.6	4.5	4.1
0.8	3 1651	4.2	"	2.5	2.5	2 2

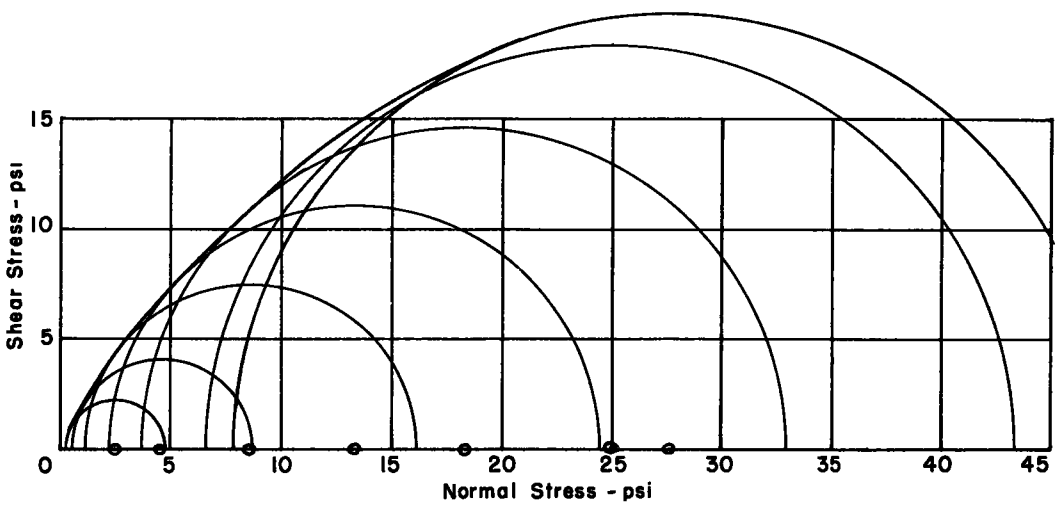


Figure 12.

14 INCH LEVEL, ρ = 100 psi., μ = 0.5, a = 5.6", F_s = 0.81

$\widehat{zz}$	ρ/a	$\widehat{\rho\rho}$	Z/a	Center $\frac{\widehat{zz} + \widehat{\rho\rho}}{2}$	S/2	Radius S/2 x 0.81
20	0	0.8	2.5	10.4	9.6	7.8
16.8	0.7813	1.8	"	9.3	8.7	7.0
13.3	1.2187	2.8	"	8.0	7.6	6.2
6.2	2.1658	4.0	"	5.1	5.0	4.1
3.6	2.7505	3.8	"	3.7	3.6	2.9

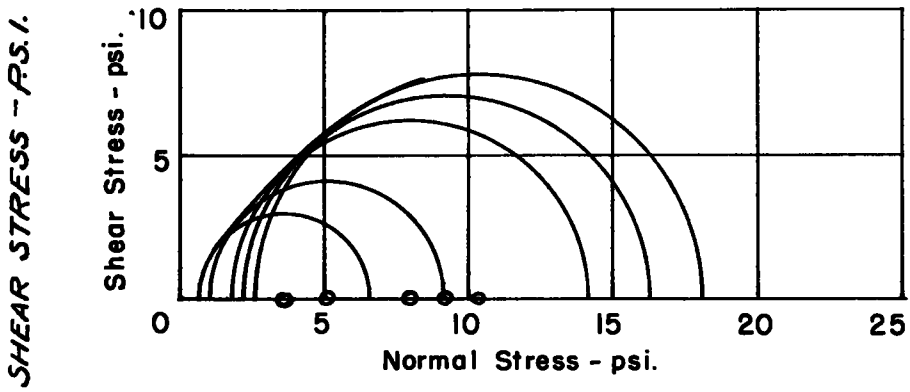


Figure 13.

28 INCH LEVEL, ρ = 100 psi., μ = 0.5, a = 5.6", F_s = 0.58

$\widehat{zz}$	ρ/a	$\widehat{\rho\rho}$	Z/a	Center $\frac{*\widehat{zz} + \widehat{\rho\rho} + 1.3}{2}$	S/2	Radius S/2 x 0.58
5.7	0	0.1	5.0	4.2	2.8	1.6
5.2	1.000	0.3	"	4.1	2.7	1.6
4.1	1.8816	0.6	"	3.7	2.3	1.3
3.5	2.8196	0.9	"	3.5	2.1	1.3
2.4	4.5010	1.1	"	3.1	1.8	1.0

*For all depths greater than 12 inches, surcharge weight in psi. = $\frac{\text{depth level inches} - 12 \times 1}{12}$

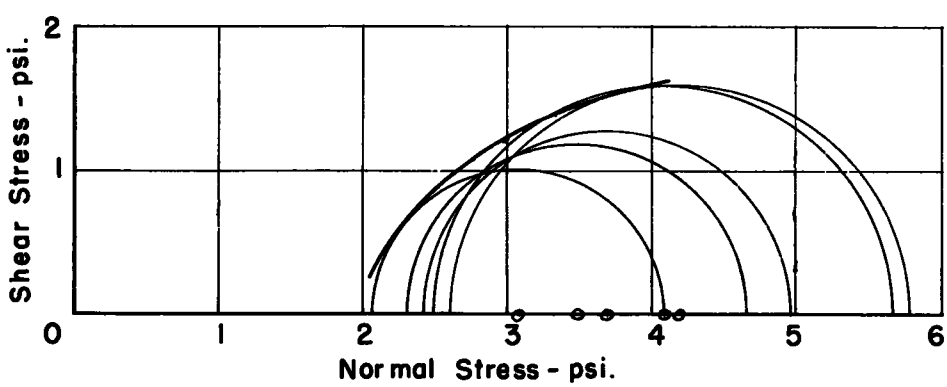


Figure 14.

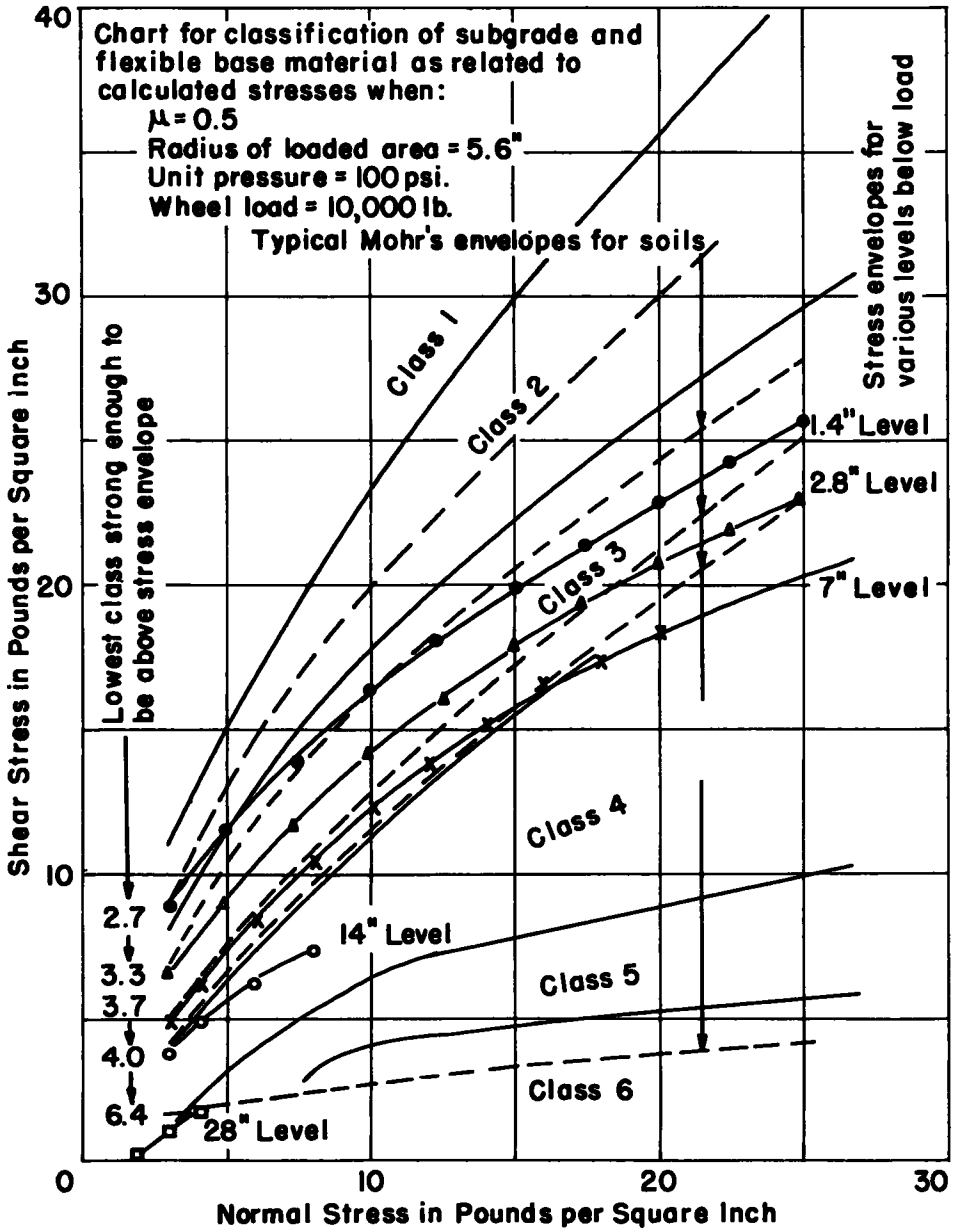


Chart for classification of subgrade and flexible base material as related to calculated stresses when:
μ = 0.5
Radius of loaded area = 5.6"
Unit pressure = 100 psi.
Wheel load = 10,000 lb.
Typical Mohr's envelopes for soils
Stress envelopes for various levels below load
Lowest class strong enough to be above stress envelope
Class 1
Class 2
Class 3
Class 4
Class 5
Class 6
1.4" Level
2.8" Level
7" Level
14" Level
28" Level
2.7
3.3
3.7
4.0
6.4
Shear Stress in Pounds per Square Inch
Normal Stress in Pounds per Square Inch
0
10
20
30
40

Figure 15.

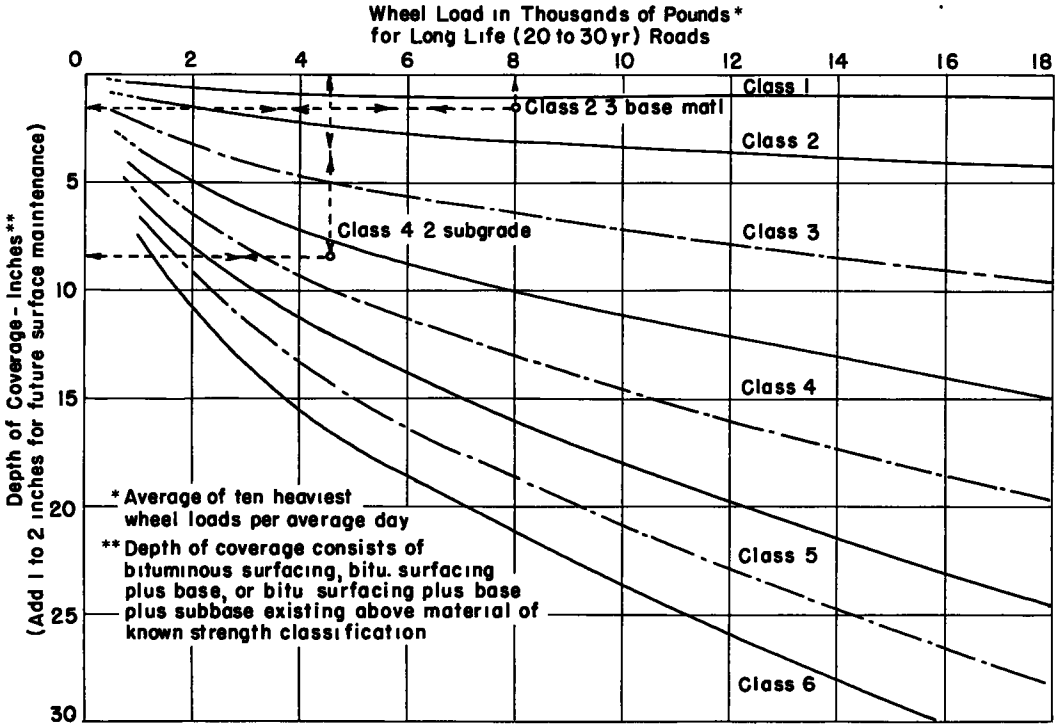


Figure 16.

Discussion

R. G. AHLVIN, Chief, Reports and Special Projects Section, Waterways Experiment Station, Vicksburg, Miss.—McDowell is to be commended for his treatment of this very difficult problem. The step from empirical to rational design methods for flexible pavements is a very large one, and McDowell has, with this paper, narrowed it somewhat.

A few minor comments appear pertinent, after which reference to an apparently remarkable correlation between the Texas Highway Department design method and the Corps of Engineers CBR design method may be of interest.

In Item 1(c) under the assumptions, the tangential stress, $\widehat{ww}$, is recognized as being the minor principal stress in a few cases. However, when Poisson's ratio is taken as 0.5 as it is here, the tangential stress is everywhere the intermediate principal stress. The assumption listed as Item 1(c) is therefore not needed.

It should perhaps be noted here that values of Poisson's ratio other than 0.5 can lead to critical maximum shear stresses (or principal stress differences) up to about 20 percent larger than those computed using the 0.5 value.

With reference to the center column of Table 3, it is not clear why the ratio of moduli, E_2/E_1, varies with depth. It is perhaps also notable that realistic ratios of moduli of two pavement layers, E_2/E_1, could be as low as about 0.2 at fairly small depths.

In Figure 4, principal stress difference contours are presented. These are the same as maximum shear stress contours $\left(\frac{\tau_{max} = \sigma_I - \sigma_{III}}{2}\right)$ except that their values are twice as large. In our stress-distribution work at the Waterways Experiment Station we have had occasion to develop very careful and, we believe, quite accurate maximum shear stress contours. A copy of these for a Poisson's ratio of 0.5 is shown as Figure A. The contours presented are only slightly different than those used by McDowell but they include smaller intervals between contours in the critical zone.

The relations expressed by the Texas Highway Department Flexible Pavement

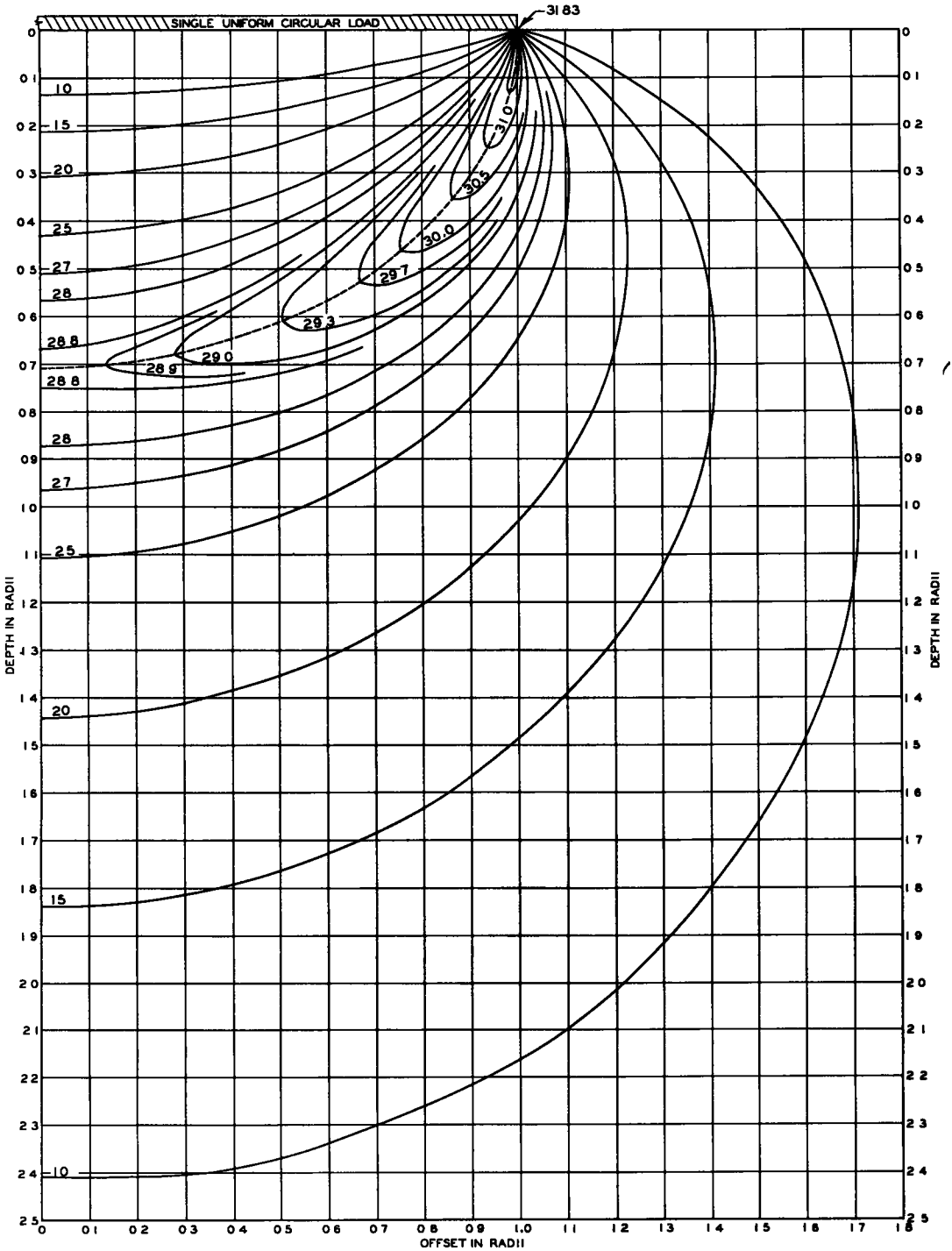

SINGLE UNIFORM CIRCULAR LOAD
31.83
31.0
30.5
30.0
29.7
29.3
29.0
28.9
28.8
28.8
28
27
25
20
15
10
27
25
DEPTH IN RADII
OFFSET IN RADII

Figure A.

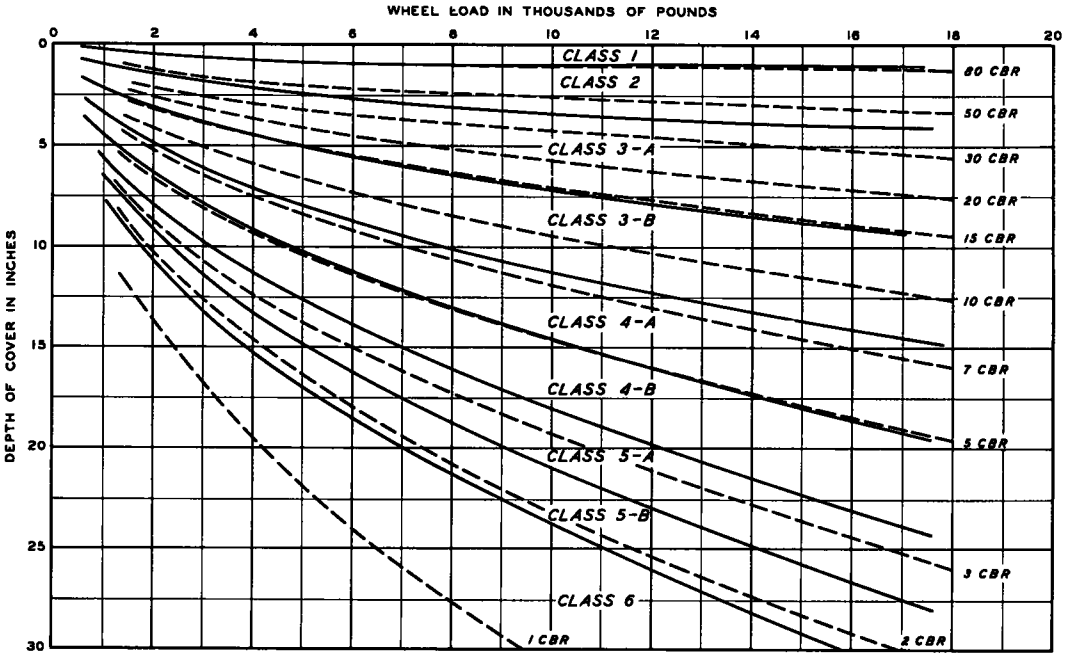


Figure B.

Analysis Chart appear to be similar in form to those of the Corps of Engineers CBR curves. Figure B shows a plot of both sets of curves for direct comparison. As can be readily seen, a remarkable parallelism exists between curves indicating a near linear relation between the Texas Highway Department class and CBR. The following table shows this relation. I have taken the liberty of adding A and B to classes 3, 4, and 5, which are divided into two parts in Figure 16 of McDowell's paper.

Texas Highway Department Class	Corps of Engineers CBR Range [a]
1	80 and above
2	38 - 80
3-A	15 - 38
3-B	7.7 - 15
4-A	5.0 - 7.7
4-B	3.4 - 5.0
5-A	2.5 - 3.4
5-B	1.9 - 2.5
6	1.9 and below

[a] The CBR relations used were those for single wheels having 100-psi. tire pressure and for capacity operation (5,000 or more coverages of aircraft traffic).

Discussion

STUART M. FERGUS, Asphalt Research Laboratory, Standard Oil Company, Cleveland Ohio—McDowell presents a method for computing subgrade stresses and comparing them with those obtained in laboratory tests of triaxial specimens. The method is developed mainly from the theory of elasticity and involves the development of a Mohr envelope for each particular wheel load. The theoretical values of the normal and shearing stresses used in plotting each Mohr diagram are obtained first by assuming that the subgrade is a semi-infinite, homogeneous, isotropic, elastic body in the usual sense and then reducing the stresses by a factor F_s. This factor is the ratio of the

maximum shearing stress in a two-layered body to that in a homogeneous body taken at a point beneath the center of the loaded circular area.

At any given depth the value of F_s depends on the ratio (E_2/E_1) of the moduli of elasticity of the two layers. The Mohr envelopes developed in this way are then plotted on the Texas Triaxial Classification Chart which presumably consists of Mohr envelopes developed from laboratory triaxial tests of soils of the several strength classifications used by the Texas Highway Department. The final step in McDowell's presentation is the relating of the wheel load stress computations to the soil strengths and the introduction of a table (Table 4) showing the depths for which the theoretical computations were made (4th column) are approximately equal to the combined thickness of pavement and base course (6th column) required by the Texas Design Curves. Computations are presented for wheel loads of 10,000 and 24,000 lb. with a surface contact pressure of 100 psi.

In a paper of this kind, dealing as it does with terms and notations with which the average engineer-reader may not be too familiar, the writer is always torn between the one extreme of explaining too little and the other of explaining what may be obvious. In the end, if the paper is to serve adequately its purpose of exposition, it is probably best to err on the side of too much rather than too little detail. With this in mind, it is suggested that the paper be amplified as follows:

1. An explanation of the source of the Mohr envelopes for the several soil strength classes of the Texas Classification Chart.

2. Some discussion of the laboratory triaxial test as presently used by the Texas Highway Department together with mention of any procedures or techniques which may be unusual.

3. Identification in some way of the terms such as "weakest class, strong enough, overstressing, etc." used in describing the strength characteristics of the subgrade, with the location, direction, and magnitude of the stresses or stress envelopes considered to be critical.

4. For most American engineers who plot the Mohr stress circle, Love's notation will be unfamiliar. A brief statement in the following form might be helpful.

$$\underline{\widehat{zz} + \widehat{\epsilon\epsilon}} = \underline{\sigma z + \sigma \epsilon} = \underline{\sigma 1 + \sigma 3}$$

$$\frac{S}{2} = \frac{\sigma 1 - \sigma 3}{2}$$

With the exception of the factor F_s, the assumption set forth in paragraphs 1 (a) to 1 (e) pertaining to the theory of elasticity are those usually made in treating the subgrade as an elastic body. It might be argued that the ratio F_s should not be applied to stresses at points away from the z-axis since a stronger top layer does not reduce all stresses uniformly. However, this seems a minor point. In this connection it might be remarked that a uniform reduction in all stresses would result if the surface contact pressure were reduced but that this would be possible only if the total load were reduced. With respect to paragraph 1 (c), a recent report[1] has shown that for the special case $\mu = 0.5$, $\widehat{ww}$ is the intermediate principal stress at all points beneath a uniform circular load.

The bases given in paragraph 8 for the development of the Texas Wheel Load Analysis Chart are also those generally accepted by pavement engineers. The proportionality of the design thickness to the square root of the wheel load is inherent also in the California Bearing Ratio method of design. It is of interest to point out in this connection that the design thicknesses given for the various wheel loads by the Texas Flexible Pavement Analysis Chart are practically identical to those required by the CBR method if the following relations are assumed:

[1] See, "Theoretical Stresses Induced by Uniform Circular Loads," TM-3 323, September, 1953. U.S. Waterways Experiment Station, Vicksburg, Mississippi.

Texas Highway Dept. Class	Calif. Bearing Ratio percent
1	100
2	80
3	40
4	8
5	3.5
6	2

A wide difference of opinion, however, can be expected on the question of the relations assumed to exist between the pattern of stress distribution in a subgrade and that in a laboratory triaxial specimen. To give one example: in the ordinary triaxial tests the lateral stress is usually held constant while the vertical stress is increased, but in a real subgrade both the lateral and the vertical stresses vary constantly between a small initial value and a maximum as the wheel load is moved on and off. It may also be pointed out that the modulus of elasticity (some writers prefer stress-strain ratio) is not a constant for soils but varies inversely with the stress. It would, therefore, seem likely that the ratio E_2/E_1 is also not a constant. One result of this variability is that computations of the strain values based on summing up or on superposition of stresses are not valid. In other words, the strain produced by a stress of 50 psi. is more than 10 times that produced by a stress of 5 psi.

All of these remarks may quite correctly be considered as quibbles which do not help to produce a satisfactory theory of flexible pavement design. McDowell's paper at the very least has merit in that it is an effort toward that goal. This reviewer, however, feels it necessary to point out that while all the present methods of design make use in one way or another of the theory of elasticity, no method has thus far been submitted which is entirely free from empiricism.

CHESTER McDOWELL, Closure—We are indebted to Ahlvin for his pertinent comments. He states that assumption item 1(c) is not needed and for all practical purposes Ahlvin is correct. However, since it is not obvious from Love's equations that 1(c) is not needed and since Tuft's tables of influence values indicate tensile values in one or two cases, I am not sure that the assumption can be deleted. Evidently, Ahlvin has proof of this point, and deletion of the assumption probably is in order. Ahlvin certainly is justified in questioning E_2/E_1 ratios shown by center column in Table 3 of the report. I can see that I failed to offer any explanation of why the values were selected. There certainly are many E_2/E_1 ratios other than those shown in Table 3. They may vary from 0.03 for soil-cement bases on subgrade soil to values far above one, the latter being cases where flexible bases are placed in rock cuts or over old concrete pavements. In these cases depth problems are not usually highly significant; but the selection of high quality base courses that are capable of resisting reflected stresses is important.

In cases where smaller E_2/E_1 ratios exist at shallower depths than those shown, it seems that shear stresses for similar depths and loadings should be less than those we would obtain by use of the values selected for use. For instance, shear stresses in subgrade under soil-cement or concrete are generally accepted to be lower than those in subgrade under flexible base.

It is believed that troubles arise when low E_2/E_1 ratios are applied because of fatigue of "slab effects". In cases where "slab effects" are attained and deflections are so low that fatigue is unlikely, some reduction of overall depths are indicated due to existence of low E_2/E_1 ratios, but we are not sure how this can be accounted for properly. Therefore, it should be stated that the E_2/E_1 ratios shown in Table 3 are preferred because shear stresses obtained by their use will be higher than those obtained by employment of lower ratios at similar depths. The decrease in ratio with depth is in the order of what we consider typical sections which utilize soil layers economically. See Table A for an example.

It is believed that the values in the table are on the safe side because shear stress

calculations obtained by their use will be higher than if lower E_2/E_1 ratios are employed at similar depths.

TABLE A

Type of Material	Depth in Terms of Z/a	Average of E_2/E_1 Ratio	Shearing Stress Factor F_S
Base E = 10,000	0 to 1	8500/10,000 = 0.85	0.93 at Z/a = 1
Good Subbase E = 8,500	1 to 2	6000/9250 = 0.65	0.84 at Z/a = 2
Fair Subbase E = 6,000	2 to 3	3800/7625 = 0.50	0.76 at Z/a = 3
Select Soil E = 3,800	3 to 5	2000/5713 = 0.35	0.58 at Z/a = 5
Subgrade E = 2000			

It is interesting to note the correlation Ahlvin shows between CBR values and our strength classifications. Although I have never attempted or seen such a correlation, it is not surprising. It should be pointed out that data for the relations shown by Ahlvin were taken from similar theories of load distribution, but that actual laboratory test results do not correlate so well. For instance, many of our strength Class 3 materials would have CBR values of 100 plus and this is one of the main reasons that we never did adopt the CBR method. We are just as interested in knowing what to build our bases out of as we are to know how thick they should be. So far as we are concerned, mold restraint is too critical in case of the CBR test when used to test base materials.

Fergus's comments certainly are of assistance in amplifying and explaining parts of the report which are not clear to the reader. He suggests that four items be amplified. It should be explained that during the author's presentation of a paper (see report in HRB Research Report 16B) covering Items 1, 2 and 3 that members of the HRB Flexible Pavement Committee requested that the report be amplified by submitting comments on the type of stress analysis considered pertinent to the Texas triaxial method of flexible pavement design. It is not surprising that this report within itself appears to be rather incomplete; however, if read in conjunction with reports given in Research Report 16-B, Bulletin 93 and Volume 26 of the Proceedings of the Highway Research Board, it will be much clearer to the reader.

Fergus correctly points out that the basis for development of the Texas Wheel Load Chart is the same as that used by the CBR method, but it is not safe to assume that his tabulated correlation between CBR and strength class exists when actual tests are made. In fact some class 3 materials containing aggregate have CBR values of 100 plus. Strength class 3 materials containing small amounts to no aggregate, such as fine sandclays, may have CBR values as low as 15. Although certain theoretical aspects of the problem correlate, it does not necessarily follow that laboratory test results show any such correlation between CBR and Texas strength classes.

It is gratifying to note that Fergus is in agreement with Ahlvin relative to the unimportance of assumption 1(c).

Fergus points out the difficulties encountered in comparing modulus of elasticity from triaxial tests to that which develops under wheel loads. It is admitted in this instance that we are dealing with very elusive or variable sets of values and we do not contend that the two sets of modulae are necessarily similar. Although such E values may be dissimilar, it does not follow that the E_2/E_1 ratios derived from triaxial tests

are necessarily different from those produced by wheel loads because in all cases each E value used has been obtained by the same procedure.

It is admitted that the Texas Triaxial Method of Flexible Pavement Design has many empirical portions in it without which the method would not function. The parts covering testing technique of the Texas method are perhaps of more value than the theoretical parts such as are being discussed here. It is doubtful if any process or method has ever been developed from theory to practical application without some empirical steps being taken. In order to utilize methods of pavement design, it seems desirable that they contain well balanced portions of theoretical and empirical steps. Considerable differences of opinion will arise as to how these portions shall be balanced.

We are indebted to Ahlvin and Fergus for their constructive and informative discussions. They will be of assistance as developments are made in our pavement design methods.

Design, Construction and Evaluation of Heavy Duty-Runways

W. H. CAMPEN and J. R. SMITH
Omaha Testing Laboratories

This paper reports the results obtained in the construction and evaluation of two flexible-type test sections constructed for estimation of the thicknesses of subbase and base required to handle 50,000-lb. wheel loads on airport runways in two widely separated localities.

In evaluating the layered system the subgrade, subbase, and base were tested successfully, with a plate having an area of 300 sq. in. The data obtained are arranged to show the strength imparting power of the subbase and base in pounds per square inch per inch of thickness.

The authors believe that the paper will prove of interest to the designers of flexible pavements, because it is shown that the required thicknesses may be estimated fairly close before the test sections are constructed, if subgrade strength characteristics and quality of superimposed layers are known.

● ABOUT 3 years ago our laboratory was engaged to design, construct and evaluate two flexible-type test sections in connection with the construction of heavy duty runways on two widely separated airports. We are presenting a report on the results obtained for two principal reasons: first, because they illustrate how the load carrying capacity of layered systems may be estimated by considering the strength of the subgrade, and the strength-imparting power of superimposed layers; and, secondly, because they should prove to be a small contribution toward the solution of the flexible-pavement-design problem.

Usually test sections are constructed and evaluated prior to the preparation of plans and specifications. However, in the two cases under consideration the usual procedure was not followed due to lack of time and suitable weather conditions. It became necessary therefore, to estimate minimum thickness requirements, and to make provisions in the plans and specifications for additional thickness, should it be required. The provisional additional thicknesses were to be dependent on the evaluations of the test sections which were to be constructed during grading operations.

The runways on these projects were to be designed to handle 50,000-lb. wheel loads on single tires inflated to 150 psi. Our client instructed us to assume that layered systems capable of supporting 50,000 lb., at a deformation of 0.2 inch, on a circular plate having an area of 300 sq. in. would perform satisfactorily. Our assignment, therefore, was to estimate the thicknesses of subbase, base and asphalt wearing surface required to obtain the specified resistance. In estimating the thicknesses of superimposed layers one must know the strength of the subgrade, and the quality of the layers to be superimposed. The strength of the subgrade is most important because it controls the rate of load carrying capacity increase of the superimposed layers. The quality of the superimposed layers is important also because it determines the magnitude of the rate of strength increase.

In the two cases under consideration the subgrade strengths were not known. However, an adequate soil survey had been made on the runway areas of both projects and on borrow pits to be used for embankment. The data obtained enabled us to estimate the load carrying capacity of the finished subgrade.

Project A is located in the plains of the Midwest. Practically all the soils encountered on the runway areas were gray clays. The moisture content was comparatively low on the upper portion but high at 6 to 9 feet below the surface. Based on our general knowledge of subgrade soils, we estimated that this subgrade would have a load bearing value of 60 psi., or a load carrying capacity of 18,000 lb. on the testing plate. In mak-

TABLE 1

PROPERTIES OF MIXTURES USED IN COMPONENT PARTS OF TEST SECTIONS

PARTS OF TEST SECTIONS

Properties	Component Parts					
	Project A			Project B		
	Subgrade	Subbase	Base	Subgrade	Subbase	Base
Gradation						
% Passing Sieve 1½"		100	100			
% Passing Sieve 1"		98	98			100
% Passing Sieve ¾"		81	81			98
% Passing Sieve ⅜"		66	66			68
% Passing Sieve #4		47	47		100	54
% Passing Sieve #10		35	35		98	38
% Passing Sieve #40	100	16	16	100	49	19
% Passing Sieve #200	97	7	7	82	21	9
Atterberg Limits						
Liquid Limit	56	21	21	49.7	23.5	22
Plasticity Index	38	5	5	31.7	9.1	3.5
Density - pcf.						
[a] M. L. D.	115.0	141.0	141.0	112.7	130	140
% M. L. D. in Test Sect.	92.7	98.2	100.1	94.8	98.4	99.1
Moisture Content - %						
Optimum	15.0	6.2	6.2	15.5	8.0	6.0
In Test Section	19.8	6.0	6.3	17.7	9.5	7.8
C. B. R. [b]						
% M. L. D.						
100		100	100			
98					89	100

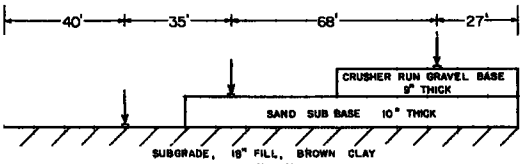
[a] M. L. D. - Maximum Laboratory Density
[b] C. B. R. - California Bearing Ratio

ing the estimate, it was assumed that embankment would be placed at a density of at least 90 percent, and that the upper 6 inches of the subgrade would be compacted to a density of at least 95 percent of maximum laboratory density obtained by the Standard Proctor Method, as modified by the U. S. Engineers.

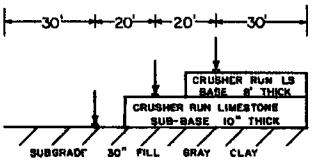
Project B is located in the foothills, on the east side of the Rocky Mountains. The prevailing soils were clays also, but because of the dryness of the underlying soils, we estimated a load-bearing value of 80 psi. which is equivalent to a load carrying capacity of 24,000 lb. on the testing plate.

Before estimating the strength imparting power of the subbase and base layers, it should be pointed out that the subgrades on these projects are classed as strong, and consequently, the rates of strength increase would be expected to be high.

Having assigned load carrying capacities to the subgrades we then proceeded to estimate the rate at which this value might be increased by the addition of subbase and base. On Project A a well-graded crushed limestone mixture, produced from rock of high quality, was to be used both for subbase and base. This mixture showed a CBR of over 100 when compacted to a density of 98 to 100 percent. We, therefore, assigned to it a strength imparting power of 4.5 and 5.0 psi. per inch of thickness, respectively, as subbase and base. This is equivalent to an increase of load carrying capacity of 1,350 and 1,500 lb. per inch of thickness, respectively, on the 300-sq.-in. testing plate.



PROJECT "B" TEST SECTION



PROJECT "A" TEST SECTION

Figure 1. Cross-sections of project test sections.

On Project B a subbase consisting of a mixture of sand and soils, and a base consisting of crusher run gravel, also of high quality, were to be used for subbase and base respectively. Both of these mixtures showed high CBR values, but the base mixture was superior. Based on these strength tests, we assigned a strength imparting power of 4 and 5 psi. per inch of thickness, respectively, to the subbase and base mixtures. The corresponding increases in load carrying capacity on the 300-sq.-in. plate is 1,200 and 1,500 lb. per inch of thickness.

After the subgrade strengths and the strength imparting power of the subbases and bases had been estimated, we proceeded to estimate the thicknesses required. First of all it was decided to use at least 8 inches of base on both projects. On project A the carrying capacity of the subgrade had to be raised from 18,000 to 50,000 lb., or 32,000 lb. Since the upper 8 inches was assumed to increase the load-carrying capacity 12,000 lb. (8 x 1,500) the subbase must increase the load-carrying capacity 20,000 lb. (32,000 - 12,000). Therefore, 14.8 inches of subbase was required (20,000/1,350).

On Project B the subgrade carrying capacity had to be raised from 24,000 to 50,000 lb. or, 26,000 lb. and, since it was desired to use eight inches of base, this thickness would increase the carrying capacity 12,000 lb. (8 x 1,500). The subbase therefore would have to contribute an increase of 14,000 lb. (26,000-12,000). The thickness required would be 11.7 inches (14,000/1,200).

Based on the foregoing estimates and calculations, we recommended 23 inches of the rock mixture for subbase and base for Project A, and 12 inches of subbase and 8 inches of base for Project B.

It will be noted that in estimating the thickness of superimposed layers the effect of asphaltic concrete was not considered, although a 3-inch mat was specified for the projects. This procedure is consistent with our usual practice. We prefer to provide for the desired load carrying capacity at the top of the base. The asphalt mat is considered primarily as a wearing surface possessing durable qualities and sufficient stability to resist shoving and rutting. Whatever strength imparting power it may possess is considered as a safety factor.

After our client considered our recommendations, the following thicknesses were selected by him: for Project A, 18 inches of subbase and base and 3 inches of asphaltic concrete; for Project B, 10 inches of subbase, 9 inches of base and 3 inches of asphaltic concrete. The test sections were constructed accordingly, except that no asphaltic concrete was added. The test sections were compacted with the same equipment which was later used to construct the runways. Care was taken to maintain optimum moisture conditions.

Cross-sections of the two test sections are shown in Figure 1. Test Section A had a width of 25 feet and B, 20 feet. The plate tests were made along the longitudinal center lines.

Some properties of the subgrade soils and the subbase and base mixtures are given in Table 1. The strength test results are indicated on the load-deformation curves in Figure 2. The data on these curves show that at a deformation of 0.2 inch, the layered systems supported the following loads:

	Project A	Project B
Top of subgrade	20,400 lb.	26,000 lb.
subbase	32,100 lb.	38,000 lb.
base	44,600 lb.	50,500 lb.

From these values the strength imparting power of the subbases and bases was calculated. On Project A, 10 inches of subbase increased the load-carrying capacity of the subgrade 11,700 lb. or 3.9 lb. per sq. in. per inch of thickness. Eight inches of base increased the load carrying capacity 12,500 lb. or 5.2 psi. per inch of thickness. On Project B, the strength imparting power per inch of thickness figures out to be 4.0 and 4.6 psi., respectively, for the subbase and base.

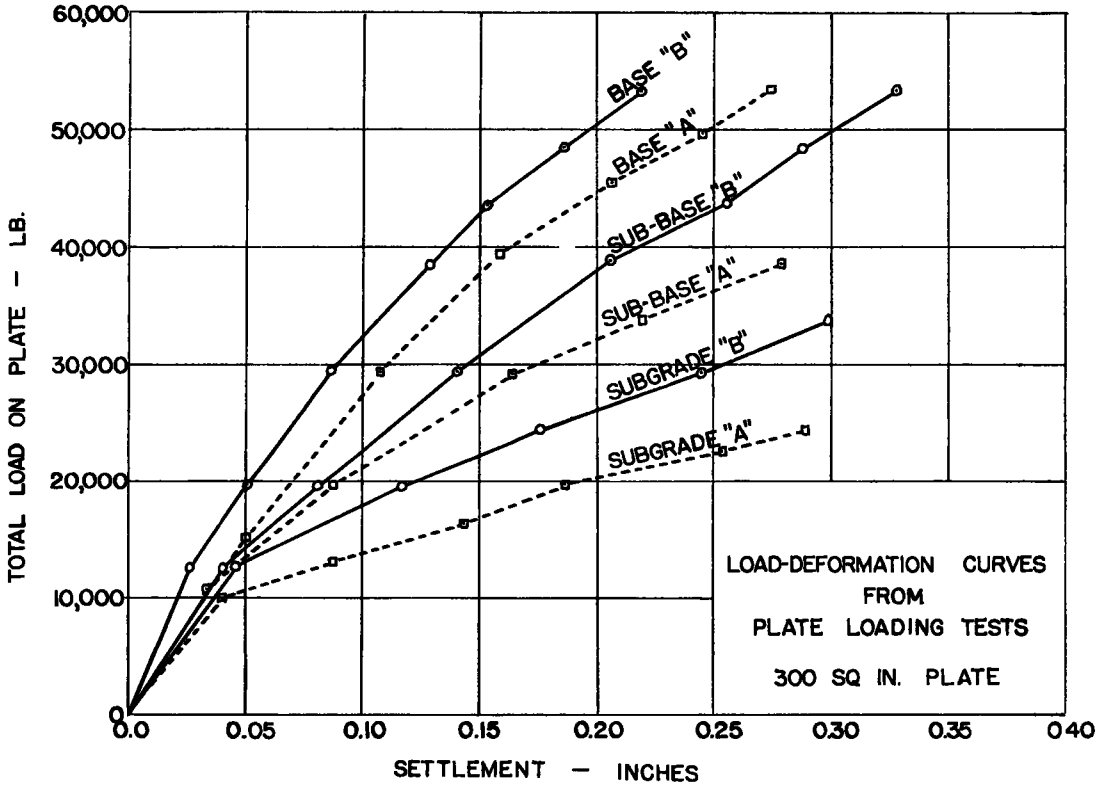


Figure 2.

Table 2 has been prepared to show a comparison between the estimated load-carrying capacities and those actually measured. The data in Column X show the measured and estimated load-carrying capacities of each test section as constructed. The data in Column Y show what the load-carrying capacities would have been if the recommended thicknesses had been used. In calculating these values, the measured subgrade strength was used, and it was assumed that the rates of strength increase determined in the sections would apply.

TABLE 2

LOAD-CARRYING CAPACITIES OF TEST SECTIONS

	Project A		Project B	
	(X)	(Y)	(X)	(Y)
Estimated LCC[a] - top of subgrade	18,000		24,000	
Measured LCC - top of subgrade	20,400		26,000	
Estimated LCC - top of subbase	31,500		36,000	
Measured LCC - top of subbase	32,100	37,900	38,000	40,400
Estimated LCC - top of base	43,500		49,500	
Measured LCC - top of base	44,600	50,400	50,500	51,400

[a] LCC refers to Load-Carrying Capacity.

Table 3 was prepared to show a comparison between the estimated and measured rates of strength increase of the subbase and base on both projects.

TABLE 3

RATES OF STRENGTH INCREASE

	PROJECT A	PROJECT B
Estimated rate of strength increase[a], subbase	4.5	4.0
Measured rate of strength increase, subbase	3.9	4.0
Estimated rate of strength increase, base	5.0	5.0
Measured rate of strength increase, base	5.2	4.6

[a] PSI per inch of thickness:

The following observations are made from the data in Tables 2 and 3: (1) We underestimated the subgrade strength on both projects. (2) The rate of strength increase in Project A was estimated too high for the subbase, but too low for the base. However, the weighted average estimated rate of 4.7 psi. for the total superimposed thickness compares fairly well with the 4.4 psi. obtained on the test section. In Project B, the estimated rate was the same as that obtained on the subbase test section but was lower on the base. The weighted average estimated rate was 4.4 psi. and the rate obtained on the test section was 4.25 psi. (3) If the thicknesses recommended had been used, the load carrying capacity of the test section on Project A would have come to expectations, but the test section on Project B would have been slightly over designed.

In connection with the evaluation of the two sections, we wish to compare the observed rates of strength increase with those determined by Palmer and Thompson during an extensive survey of airport runways. They reported the findings in 1947 in a paper entitled "Pavement Evaluation by Loading Tests at Naval and Marine Corps Air Stations." The runway pavements were evaluated with a plate having an area of 707 square inches and at a deformation of 0.2 inches. Tests were run on top of the pavements and on the subgrades beneath them. They analyzed the data by plotting subgrade versus pavement load carrying capacity.

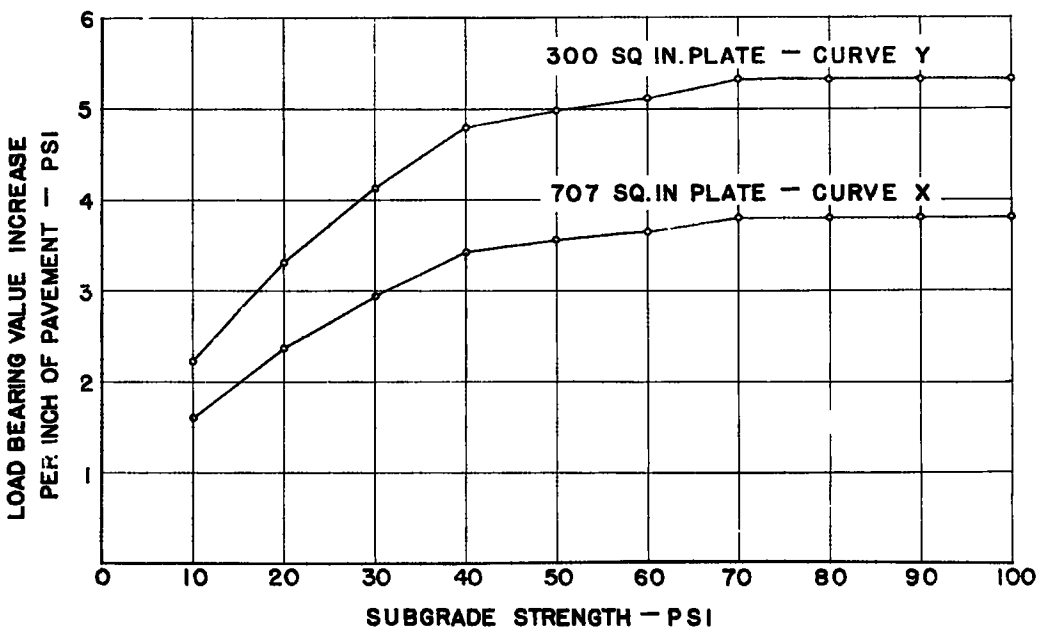


Figure 3. Strength-importing power of superimposed layers in flexible pavements.

In discussing the paper; we pointed out that the increase of load carrying capacity contributed by superimposed layers can be expressed in psi. per inch of thickness. Now we wish to point out that the rates of strength increase obtained on the test sections are somewhat less than those reported by Palmer and Thompson. In order to do so we have constructed two curves shown on Figure 3. Curve X is the same as that used in the discussion and shows rates of strength increase with a 707 sq. in. plate. Curve Y shows what the rates of strength increase would have been if a 300-sq.-in. plate had been used. In constructing Curve B, it was assumed that a layered system shows 40 percent more strength with the smaller plate. This relationship is approximately correct as has been shown by both Norman W. McCleod and ourselves.

In order to compare the rates of strength increase, we need to refer to Curve Y. On this curve the rates of strength increase are 5.25 and 5.30 psi. for subgrade strengths of 68 and 87 psi., respectively. These subgrade strengths were measured for Projects A and B, and the rates of strength increase turned out to be 4.50 and 4.3 psi., respectively. The comparison indicates that the rate of strength increase obtained in the test sections is approximately 83 percent of that reported by Palmer and Thompson. This difference may be an indication that the flexible pavements become stronger during the application of traffic.

In concluding this report we wish to add that we realize that the data presented is rather meager. However, it should serve its purpose as an example of what can be done in a practical way to design flexible pavements. We realize also that our method of design is based on a personal knowledge of the following factors: (1) the effect of subgrade strength on the strength imparting power of superimposed layers; (2) the effect of the quality of superimposed layers on the magnitude of the rate of strength increase; and (3) the effect of size of testing plate.

Discussion

RAYMOND C. HERNER, Chief, Airport Division, CAA Technical Development and Evaluation Center—We who are limited largely to work in the laboratory are indebted to men like Campen and his associates who are willing to take the time to give us information on their design and construction activities. This information is helpful in bridging the gap between the laboratory and the field.

There is one point in the Campen-Smith paper which seems to deserve special comment. In several instances it refers to the strength-imparting power of the material in a manner which might imply that this is a definite value inherent in the material without regard to any other factor. Our tests on the load-transmission apparatus have shown that two materials may be nearly equal in performance when used in the upper layers of a pavement but may vary considerably when used at greater depths where they are confined by superimposed layers.

In building up a trial pavement as described in this paper, we may find a large increase in strength on addition of a layer of material, due not alone to the inherent qualities of the added material but due also to the added confinement given a pervious layer which is of a nature that it will benefit from such confinement. This point should be kept in mind when trying to translate experience from one location to another location where the paving materials and subgrade have different characteristics.

HOWARD NUNEZ and CHARLES W. JOHNSON, Materials and Tests Division New Mexico State Highway Department—While, in all probability, the methods employed by Campen and Smith in the design of surface thicknesses for the two subject runways are quite valid, it is felt that there has been omitted from the report a great amount of information necessary to substantuate the decisions reached. What, for instance, was the nature of the knowledge that led to the estimate of 60-psi. bearing value for the subgrade on Project A? Why could not this estimate have as easily been 65 psi.—an estimate much nearer the true value?

On studying the report, one cannot help but notice the apparent discrepancies between

the estimated and measured load-bearing capacities. While it may be true that the weighted averages of the estimated rate of strength increase for the base and subbase materials for both Project A and B agree fairly close with the weighted averages of measured strength increase, it must be borne in mind that the design was not based upon average values, but rather upon definite assigned rates of strength increase for each material.

In establishing the values of 4.5 and 5.0 psi. per inch of thickness as the strength factor for, respectively, the subbase and base on Project A, the authors must have relied on some information or knowledge not presented in the paper. Also, if, as the authors suggest, an increase in load carrying capacity is caused by superimposed layers of material, it would appear that a different strength factor would be required for each lift in which material was placed. Either that is indicated or a single strength factor could be used for designing a thickness of surfacing to be built entirely of the same material, regardless of the layer in which it was to be placed.

As a further point of interest, it is noted that both runways were to be designed to handle 50,000-lb. wheel loads, yet all bearing tests were made on the top of the base course and any increase in bearing capacity due to the specified three inches of asphaltic concrete was considered to be a factor of safety. Would it not probably result in a more economical design to employ a safety factor in each layer and to include the asphaltic concrete in the design of the layered system. It would be interesting to see results of plate-bearing tests conducted on top of the asphaltic pavement of the test sections.

Flexible-Pavement Design with Cone Device

W. A. WISE, Materials Engineer
North Dakota State Highway Department

THIS paper describes the procedure being used in North Dakota for the design of flexible pavements in which the necessary thickness of subbase material underlying the standard base and pavement section is determined. Subbase material is called for where the cone bearing value of the subgrade is less than 400 psi. Since the cone tests are made generally, when the condition of the subgrade is good, the design of the pavement may not be adequate during the spring thawing period, and present practice in North Dakota provides for load limitation at this time.

● THE cone device has been in use by the North Dakota State Highway Department for more than 15 years. Its use provides a fairly rapid and simple method for determining the bearing power of fine textured subgrade soils. The equipment is simple, compact and portable.

CONE DETAILS AND DIRECTIONS FOR USE

The steel cone is 4 inches long, having an angle at the tip of 15 deg. 30 min. The cone tip is mounted downward on a ¾-inch round steel shaft that is inserted in a small steel frame. Figure 1 shows the details of the cone bearing device. The method of determining the bearing power of soil with the cone device is given in the Appendix.

FIELD OPERATIONS

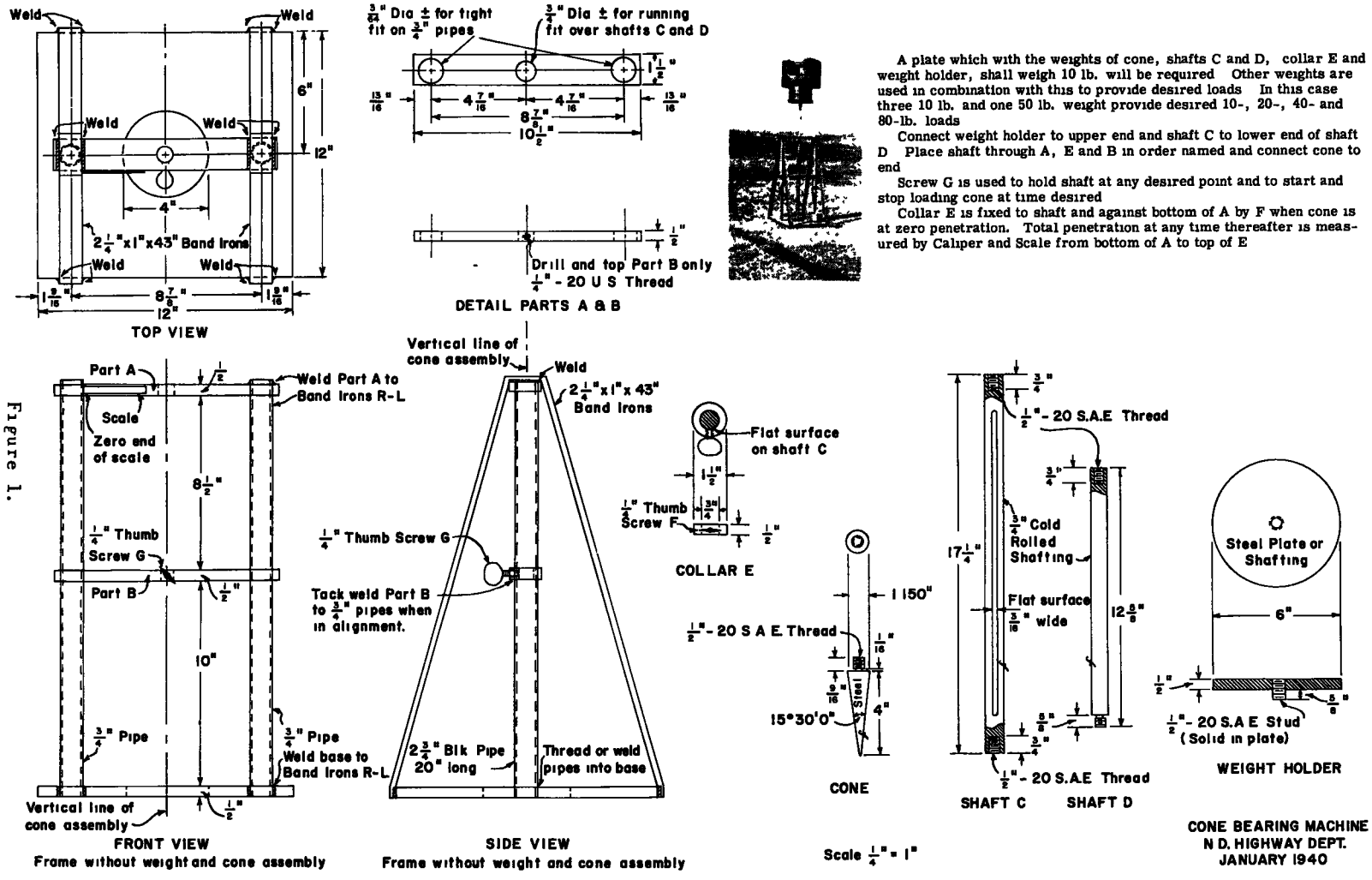
First the bearing is taken at an elevation 3 inches below the subgrade surface; then at points 9, 15, and 24 inches below the subgrade surface. These four readings are averaged to determine the bearing power of the subgrade at the test point. The bearing result is an average rating for the top 24 inches of the subgrade. This method was developed through practice and experience in order to arrive at an accurate reading having some factor of safety. Originally only the crust reading of the subgrade surface was tested and accepted as the true bearing power. Such bearings generally were "high" which often resulted in the selection of a flexible pavement section of insufficient thickness for efficient performance.

Now the four subgrade readings are taken as described in order to determine a more practical bearing power. In this procedure the high subgrade crust value is included but does not over influence the final result. This method of average rating is also believed practical because of the variable weather conditions which seriously affect subgrade bearing strengths. No scientific method has been devised to allow for these changes during the season. Therefore, the judgement and experience of the engineers associated with the cone bearing work largely govern. Their design recommendations receive great consideration.

Bearing tests are taken as often as required to represent the project involved and the soils encountered. Under normal conditions one subgrade test station per mile is usually sufficient. At each test station three separate test holes are utilized on the right or left side of the centerline by random choice. By the method of average bearings for the top 24 inches of the subgrade and the utilization of engineering judgement, a flexible pavement section is selected which is fairly uniform in thickness. Necessary reinforcements for limited distances are made by use of additional pit-run granular subbase material. These reinforcements are placed before construction of the standard base and pavement section begins. Typical standard flexible pavement design sections are shown in Figure 3.

DESIGN PROCEDURE

By means of the chart in Figure 2 the required total base and surface thickness is selected to suit the subgrade bearing value. However, the standard section of 9½ inches



A plate which with the weights of cone, shafts C and D, collar E and weight holder, shall weigh 10 lb. will be required Other weights are used in combination with this to provide desired loads In this case three 10 lb. and one 50 lb. weight provide desired 10-, 20-, 40- and 80-lb. loads

Connect weight holder to upper end and shaft C to lower end of shaft D Place shaft through A, E and B in order named and connect cone to end

Screw G is used to hold shaft at any desired point and to start and stop loading cone at time desired

Collar E is fixed to shaft and against bottom of A by F when cone is at zero penetration. Total penetration at any time thereafter is measured by Caliper and Scale from bottom of A to top of E

Figure 1.

total base and surface thickness is used regardless if the subgrade bearing tests more than 400 psi. This procedure has proven necessary because of the extreme weather conditions in this state and also to provide a reasonable factor of safety. If the subgrade tests under 400 psi., the necessary additional thickness is provided by the use of pit-run granular subbase prior to the placement of the standard 9½-inch flexible-pavement section.

This standard pavement section consists of a 5-inch pit-run granular subbase, a 2-inch stabilized base (either gravel or bituminous as specified), and a 2½-inch asphaltic-concrete wearing course as shown in Figure 3. This pavement section, for subgrades testing 400 psi. or more, is considered suitable for a wheel load of 9,000 lb., except in the spring thawing period. During that period load limits must be applied to preserve the highway pavements until the subgrades have recovered sufficiently to carry normal loads again. Incidentally, the loss in subgrade bearing power during the spring thawing period has been determined to be approximately 50 percent by research work being conducted by the Highway Research Board Maintenance Committee No. 7. The research project's title is "Investigation of Load Carrying Capacity of Roads as Affected by Frost Action."

CORRELATION WITH OTHER METHODS

Research work developed by Norman McLeod, of Canada, as reported on Page 77 of the June 9, 1949, issue of the Engineering News-Record, indicates the following approximate correlation of subgrade test results by three standard methods:

North Dakota Cone Device	400 psi.
California Bearing Ratio	10.25
Housel Penetrometer	24 blows for 6 inches of penetration

McLeod's more recent report in the March-April 1954 issue of "County and Township Roads" indicates a flexible pavement section of 9½ inches is safe for a 9,000-lb. wheel load. This result is in substantial agreement with North Dakota's standard design practice. The bulk of the North Dakota highway system mileage falls in the medium traffic classification with less than 2,000 vehicles per day of which no more than 50 are heavy vehicles carrying maximum legal loads.

LIMITATIONS

The cone device has its limitations and must be used with judgement to be of practical value. When testing subgrades of finished pavement work, it is necessary to open a test hole through the pavement and base. The cone device rests on the pavement surface and an extended shaft is used to make the test through a small diameter hole in the pavement section. Although the test hole is carefully refilled and tamped this procedure requires time and destroys the homogeneity of the pavement section at the test point. Therefore, the cone device has its most practical use in testing fine textured earth subgrades before the next stage of construction begins.

Correlation of field and laboratory cone test results has not proven successful. Field cone tests of in-place subgrades are accepted and used. But cone tests on the same soils compacted in the laboratory have never been in consistent agreement with field tests and have considerable fluctuation. Therefore, the field tests govern.

ACTUAL PERFORMANCE PROCEDURE

Typical design procedure for new pavement, and resurfaced pavement, and a report of actual performance of pavement subgrades follow.

New Base and Surface Work

Federal Aid Secondary Project No. S-254(11) on US 13 from Wishek East in McIntosh County was placed under contract August 30, 1954. The pavement section specified was a 5-inch pit-run gravel subbase, a 2-inch stabilized gravel base and a 2½-inch asphaltic concrete surface. On high-type primary road projects, a 2-inch asphaltic concrete

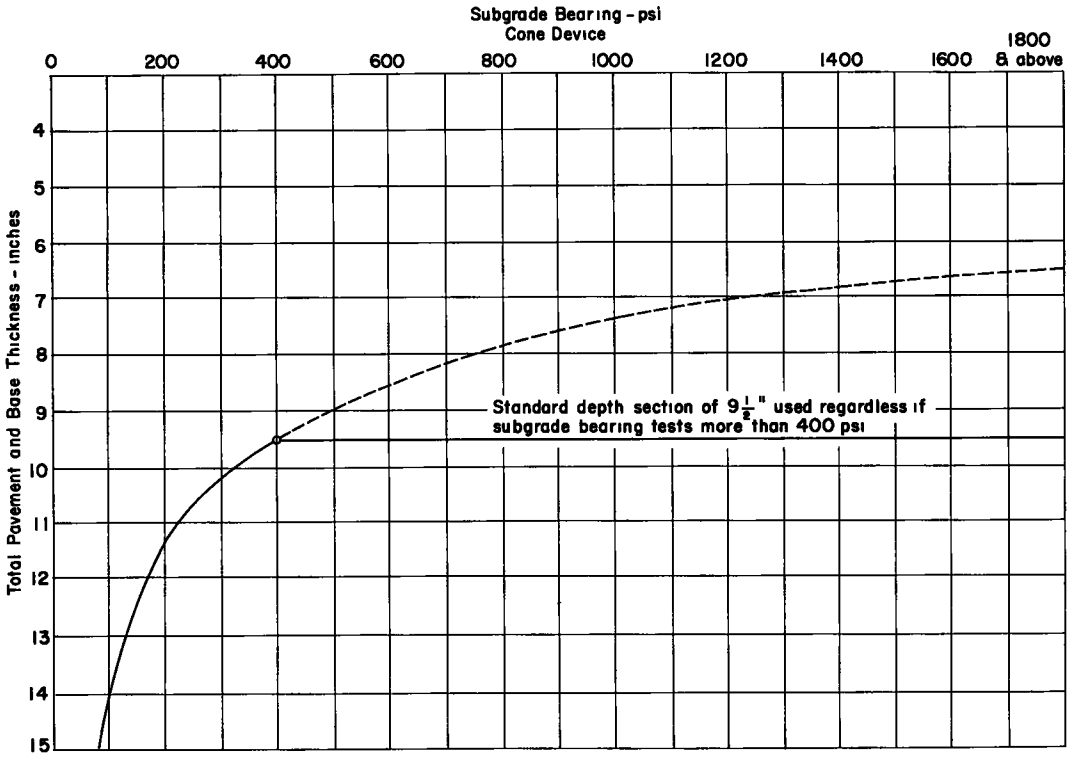


Figure 2.

base is usually substituted for the 2-inch stabilized gravel base; 10 percent additional material of the plan quantity of granular subbase was allowed for subgrade reinforcing. Such reinforcing is used on all work of this kind as the conditions require and as may be authorized by the engineer.

This project was graded in 1952 and 1,200 cubic yards of traffic service gravel were placed per mile for temporary usage until the route was paved. In North Dakota design procedure this limited amount of temporary gravel is considered to be dissipated and of no apparent structural value when the paving work proceeds. Traffic is maintained through all projects during construction.

The traffic count on this unpaved project in 1953 was 340 vehicles including 85 trucks.

Subgrade bearing tests for two typical test point stations on this project in 1954 are shown in Table 1.

In the area represented by the bearing results at Station 548+15 the engineer will increase the subbase thickness of ½ inch during construction in order that the total depth of paving section of 10 inches will meet the requirements of the design curve chart. The limits of the area involved are determined by visual inspection, by noting the amount of subgrade deflection under heavy wheel loads, and by additional cone bearing tests when considered necessary.

Leveling Course and Resurfacing Work

Federal Aid Primary Project No. F-71(5) on US 13 from Wyndmere East in Richland County was placed under contract on March 23, 1954. The current contract specified an asphaltic concrete leveling course of 1½-inch average depth. This course is used for the purpose of re-establishing proper crown and grade and also for minor structural reinforcements where major reconditioning work is unnecessary. The final wearing course specified was a 2-inch depth of asphaltic concrete. The traffic count in 1953 was 795 vehicles including 200 trucks.

The project was graded in 1939 with 700 cubic yards of traffic service gravel placed

TABLE 1

FAS PROJECT No. S-254(11)

Date	Sta.	Dist. from ℄	Subgrade Test Depth	Cone Bearing	Av. Bear. for 24 inch Depth	Remarks
		feet	inches	psi.	psi.	
			3	1341		
			9	569		
9/2/54	401+10	4 Lt.	15	586	706	
			24	329		
			3	1970		Average bearing for test station is 686 psi. O. K. for Stand. 9½-inch paving section.
			9	1025		
9/2/54	401+10	8 Lt.	15	374	948	
			24	424		
			3	559		
			9	474		
9/2/54	401+10	12 Lt.	15	235	403	
			24	344		
			3	971		
			9	496		
9/3/54	548+15	4 Lt.	15	229	481	Average bearing for test station is 347 psi. By curve chart 9.88 inch paving section required. 10 inches accepted as practical depth to use.
			24	230		
			3	606		
			9	234		
9/3/54	548+15	8 Lt.	15	224	311	
			24	178		
			3	379		
			9	234		
9/3/54	548+15	12 Lt.	15	174	250	
			24	214		

per mile for temporary usage. The original paving section was placed in 1940. It consisted of a 6-inch stabilized gravel base and a 1½-inch hot mixed bituminous surface course containing SC-4 bitumen. While this original paving section would not meet current design standards, it has given satisfactory service. It has received minor routine maintenance, periodic seal coats at approximately five year intervals, and patching at some spot failures. There is no record of the bearing power of the original subgrade but it is assumed to have been at least 400 psi. by the cone device method. To preserve the pavement section after fourteen years service it was necessary to provide for reconditioning the pavement in 1954 by adding structural strength, providing satisfactory riding quality and a new asphaltic wearing course.

Subgrade bearing tests for three typical test point stations on this project in 1954 are shown in Table 2.

The area represented by test station 129+50 had an average bearing of 322 psi. It required a paving section with a total depth of 10 inches from Figure 2. The existing base and surface depth was found to be 8¾ inches. The project plan specified the placement of a new 2-inch asphaltic surface. Total depth of section would then be 10¾ inches which was satisfactory as it exceeded the required design depth of 10 inches. In addition to this procedure a thin asphaltic leveling course was used at the Engineer's discretion as provided by the plan to satisfactorily recondition the existing surface pertaining to crown and grade prior to the placement of the final 2-inch wearing course.

TABLE 2

FEDERAL AID PROJECT No. F-71(5)

Date	Sta.	Dist. from ℄	Subgrade Test Depth	Cone Bearing	Av. Bear. for 24 inch Depth	Remarks
		feet	inches	psi.	psi.	
			3	397		
			9	292		
8/25/54	129+50	4 Lt.	15	598	359	
			24	151		
			3	266		Average bearing for test station is 322 psi. Total depth required from Curve Chart "C", is 10 inches. Existing base and surface is 8¾ inches. New 2-inch asphaltic surface O. K.
			9	No test (Grav.)		
8/25/54	129+50	8 Lt.	15	481	297	
			24	144		
			3	322		
			9	No test (Grav.)		
8/25/54	129+50	12 Lt.	15	499	310	
			24	110		
			3	275		
			9	1300		
8/26/54	187+85	4 Lt.	15	285	494	
			24	114		
			3	1340		Average bearing for test station is 457 psi. Total depth required from Curve Chart "C" is 9½ inches. Existing base and surface is 8¼ inches. New 2-inch asphaltic surface O. K.
			9	554		
8/26/54	187+85	8 Lt.	15	338	585	
			24	108		
			3	303		
			9	474		
8/26/54	187+85	12 Lt.	15	254	293	
			24	140		
			3	189		
			9	302		
8/26/54	230+25	4 Rt.	15	735	379	
			24	290		
			3	268		Average bearing for test station is 336 psi. Total depth required from Curve Chart "C" is 10 inches. Existing base and surface is 8¼ inches. New 2-inch asphaltic surface O. K.
			9	292		
8/26/54	230+25	8 Rt.	15	692	377	
			24	255		
			3	465		
			9	186		
8/26/54	230+25	12 Rt.	15	163	252	
			24	194		

The area represented by test Station 187+85 with an average bearing of 457 psi. required a paving section with a total depth of 9¼ inches from Figure 2. Therefore, a 9½-inch section was required as that is the standard minimum depth of paving section used regardless if the subgrade bearing strength tests more than 400 psi. The existing

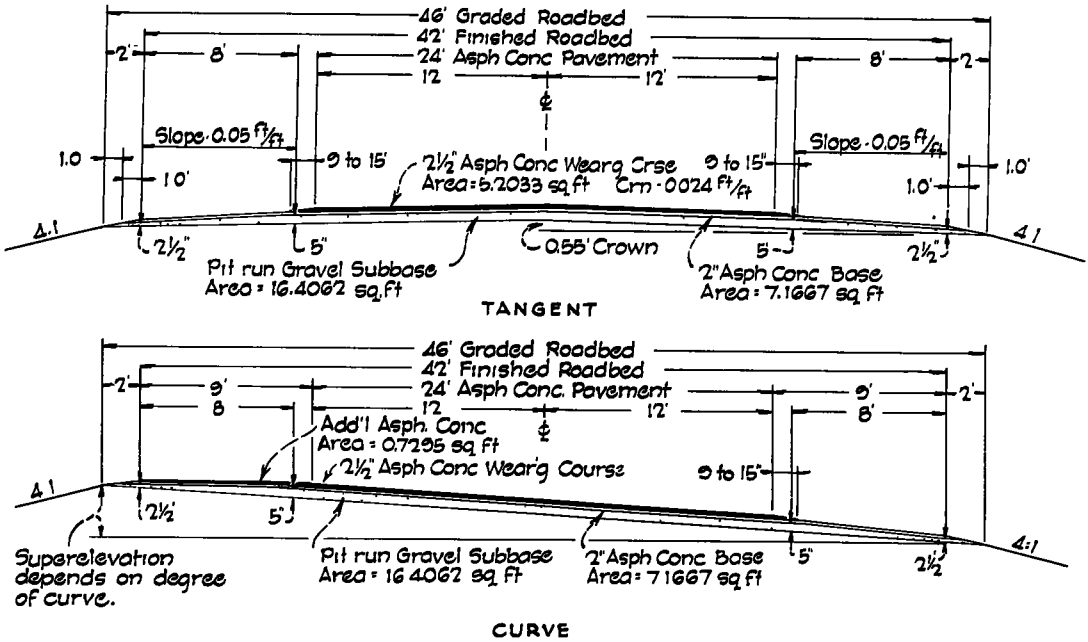


Figure 3. Typical flexible pavement sections, North Dakota State Highway Department.

base and surface depth was 8¼ inches. The project plan specified a new 2-inch asphaltic wearing course plus necessary leveling course to restore crown and grade. This resulted in a total depth of more than 10¼ inches which was greater than the 9½-inch section required and was therefore satisfactory.

The area represented by test station 230+25 with an average bearing of 336 psi. required a paving section with a total depth of 10 inches from Figure 2. The existing base and surface depth was found to be 8¼ inches. The project plan specified the placement of a new 2-inch asphaltic surface. Total depth of section would then be 10¼ inches which was satisfactory as it slightly exceeded the required design depth of 10 inches. A thin leveling course was also used as specified by the plan to satisfactorily recondition the crown and grade.

Occasionally on this type of project the total depth of section required by the design curve may be 1 inch to 2 inches greater than that which will be obtained by adding only the new 2-inch wearing course and a thin leveling course. In such a case the additional depth required is obtained by use of an increased amount of leveling course for the area involved. The plan quantity of leveling course material is provided for use in variable thicknesses to properly recondition the existing surface. This result is generally obtained within the limits of the quantity shown on the plan for this purpose. Overruns in the leveling course quantity sometime occur during construction operations because of unforseen deterioration of the existing paving section after the design is completed and before construction work begins. Substantial recoveries of such overruns are made by laying the final wearing course slightly thinner than the required plan thickness. The specifications accept a wearing course that is at least ⅞ of the required plan thickness. This procedure has proven practical because the required total depth of paving section is obtained due to the extra depth of leveling course used. In this practical manner the project is completed satisfactorily and an overrun in total construction cost rarely occurs.

Subgrade Performance of Completed Projects

Subgrade performance history of four permanent test points for projects of varying types and ages follows. The top 24 inches of the subgrades have been evaluated in the standard manner described in this report by means of cone tests each year from the

spring to the fall season. The first tests were made in the fall of 1948. The results of these four test points are typical of the performance information being obtained from a total of twelve test points. In general the construction subgrade bearing values were much greater than the minimum requirements of 400 psi. for the standard 9½-inch flexible pavement section now used. During each year of service there has been a rather continuous loss in subgrade bearing power until some current results have become alarmingly low. This progressive decline in strengths has occurred because after the annual loss in bearing power during the spring thawing periods, the subgrades have not fully regained their former strengths in the subsequent summertime recovery periods. The minimum point at which the subgrade strengths may be expected to level off without further permanent losses has not been determined.

No subgrade bearing results for 1954 have been compiled for release at this time for the four typical test points being reported on herein. Average bearing results for these four points from the fall of 1948 to the fall of 1953 follow:

A. Test Point No. 7 is on US 10 east of Sterling in Burleigh County. The project was graded in 1946 and surfaced in 1947. In the grading operation a 3¾-inch temporary gravel surface was placed. This was largely dissipated and incorporated into the subgrade by weather and traffic action by the time construction of the pavement section began. The pavement section consisted of a 5-inch stabilized gravel base and a 2½-inch hot mixed surface containing SC-4 bitumen. In 1953 the traffic count was 1645 vehicles including 410 trucks.

Average bearing values for the top 24 inches of the subgrade for this test point from the fall of 1948 to the fall of 1953 follow:

Project	Station	Pave. Const.	Actual Depth of Sect.	Year Tested	Average Spring Value	Average Fall Value
			inches		psi.	psi.
SN-FAI 306(25)	336+00	1947	7.5	1948	No Tests	656
			7.75	1949	413	650
			6.9	1950	299	557
			7.0	1951	357	385
			7.1	1952	158	269
			7.75	1953	186	277

The design curve was revised since this project was constructed. Therefore, the pavement section on this project is less than that required by the design curve now in use. It will be noted that the subgrade bearings have steadily declined since the original construction and since 1951 have been seriously low. The performance of this section of pavement is being watched carefully but thus far no visible distress has been detected. The deferment of anticipated deterioration may be due to the subgrade crust lying above the first cone test elevation at the 3-inch depth having been sufficiently upgraded by the original temporary gravel surface placed in 1947 to prevent any noticeable deterioration to date.

B. Test Point No. 8 is on US 10 east of Menoken in Burleigh County. This project was graded and paved in 1947. With the grading work a temporary traffic service gravel course was placed at the rate of 1,200 cubic yards per mile. The paving section consisted of a 5-inch gravel subbase, a 2-inch stabilized gravel base, and a 2½-inch asphaltic wearing course. The traffic count in 1953 was 1955 vehicles including 490 trucks.

Average bearings for the top 24 inches of the subgrade for this test point from the fall of 1948 to the fall of 1953 follow:

Project	Station	Pave. Const.	Actual Depth of Sect.	Year Tested	Average Spring Value	Average Fall value
SN-FAI			inches		psi.	psi.
174B	26+00	1947	9.25	1948	None	746
			9.3	1949	343	623
			9.0	1950	281	512
			9.0	1951	343	390
			9.0	1952	230	405
			9.1	1953	308	427

This test point also indicates a steady loss in subgrade bearing power although the minimum maintained since 1951 approximates the 400 psi. required for the standard 9½-inch pavement section. The total depth of pavement section measures slightly less than the required 9½-inch thickness. The pavement on this project has shown progressive distress in recent years and it developed considerable map cracking. In 1954 this project was strengthened by the placement of a 2-inch road mix course containing MC-4 bitumen.

C. Test Point No. 9 is on US 10 west of Jamestown in Stutsman County. This project was graded in 1941 and a temporary traffic service gravel course at the rate of 1,200 cubic yards per mile was included in the same contract. A 5¾-inch stabilized base was placed in 1942 and sealed with MC-4 bitumen and a sand aggregate cover. Further improvement was deferred due to World War II. Then a 2½-inch hot mixed bituminous surface was placed in 1944 containing MC-3 bitumen. The traffic count in 1953 was 2,090 vehicles including 548 trucks.

Average bearing values for the top 24 inches of the subgrade from the fall of 1948 to the fall of 1953 follow:

Project	Station	Pave. Const.	Actual Depth of Sect.	Year Tested	Average Spring Value	Average Fall Value
SN-FAP			inches		psi.	psi.
279A	255+00	1944	9.75	1948	No Tests	845
			9.5	1949	481	886
			9.9	1950	514	759
			10.4	1951	577	762
			10.5	1952	609	906
			10.0	1953	352	522

This point has consistently tested higher than all other performance test points. For four years no continuous loss in bearing power was indicated. However, a sudden alarming decrease occurred in 1953. The future performance of this test point is being watched with much interest to attempt to determine plausible reasons for its actions.

D. Test Point No. 10 is on US 52 southeast of Donnybrook in Renville County. Grading work began in 1946 and was completed in 1947. In 1947 a temporary traffic service gravel course at the rate of 1,000 tons per mile was placed. The pavement section was placed in 1948 and consists of the standard 9½-inch section having a 5-inch pit-run subbase, 2-inch stabilized gravel base, and 2½-inch asphaltic wearing course. The project was resealed in 1952 with an application of RC-2 bitumen and a sand type aggregate cover. The traffic count in 1953 was 790 vehicles including 160 trucks.

Average bearing values for the top 24 inches of the subgrade from the fall of 1948 to the fall of 1953 follow:

Project	Station	Pave. Const.	Actual Depth of Sect.	Year Tested	Average Spring Value	Average Fall Value
			inches		psi.	psi.
SN-FAI 180(3)	199+30	1948	10.0	1948	No tests	662
			9.5	1949	450	533
			10.2	1950	166	257
			10.6	1951	172	237
			10.2	1952	166	231
			9.5	1953	171	241

This test point has shown a continuous loss of bearing power and up to date is the poorest point being tested. The project pavement shows progressive distress and is cracking badly. It may require prompt strengthening and reconditioning to prevent excessive deterioration.

SUMMARY

1. The cone device is rapid to use and is simple, compact, inexpensive and readily portable.
2. Its use is limited to testing the bearing power of fine textured subgrade soils.
3. Correlation of field and laboratory cone test results for the same soils has been unsuccessful.
4. The plate bearing method appears to be practical for testing a finished pavement section. For cone tests to be made a test hole must be opened through the base and pavement into the earth subgrade and the test hole refilled after the test is completed.
5. Regardless of its limitations the cone device has valuable practical usage. North Dakota expects to continue its usage in the standard manner developed by practice and experience.

Appendix

NORTH DAKOTA STATE HIGHWAY DEPARTMENT

Method of Determining Bearing Power of Soil with Cone Device

PROCEDURE OF TEST

The subsoil should first be scraped level at the point where the bearing is to be taken. The cone machine can then be set in place and the cone adjusted so that it just touches the subsoil. The collar on the cone shaft is locked in place against the top cross-piece.

The cone is now loaded with a 10-lb. load and is released slowly to prevent impact, and is allowed to settle for one minute. The cone shaft is then locked in place and the amount of penetration recorded. This is the distance between collar and top cross-piece and is measured with calipers to the nearest hundreth of an inch.

The load is then increased to 20 lb. and the cone is released slowly, allowing it to settle for another minute. The total penetration is measured. The loads are then increased to 40 and 80 lb. successively, using the same procedure.

All of the above loads include the weight of the cone and shaft.

The bearing power of the soil is expressed in psi. based on the cross sectional area of the cone at the ground line, and the load. Tables have been prepared for each load so that the corresponding bearing value is read directly for each penetration.

Theoretically, discounting friction, etc., the penetration of the 10-lb. load is ½ of the penetration of the 40-lb. load; and the penetration of the 20-lb. load is ½ of the 80. Actually, this never works without first making a correction.

Sometimes when starting a test the cone is not exactly touching the subsoil, but may have penetrated slightly, or may be above; in either case all of the readings may be too small or too large. Then the cone has a rounded point and all the readings are too small on that account. Both of the above corrections can be taken care of at the same time by adding or subtracting the same amount to or from all the readings till the penetration of the 20-lb. load is ½ of the penetration of the 80 lb.

Example:

Load Lb.	Penetration 100ths Inch	Correction Penetration	Bearing psi.
10	44	54	592
20	66	76	596 (Average
40	98	108	592 594)
80	142	152	596

Computations: 44 x 2 = 88. 98 - 88 = +10, the correction required to restore all penetrations to the correct figures. (66 x 2 = 132. 142 - 132 = +10—Check).

The bearings for each load may vary a small amount and the true bearing is taken as the average.

TABLE A

PENETRATION RESISTANCE—PSI.

10-lb. Weight on Narrow Cone

		.02	.04	.06	.08
		431,500	107,875	47,944	26,968
.1	17,260	11,986	8,804	6,741	5,325
.2	4,315	3,566	2,996	2,553	2,201
.3	1,918	1,685	1,492	1,331	1,195
.4	1,078	978	891	815	750
.5	690	638	592	550	513
.6	479	449	421	396	373
.7	352	332	315	299	283
.8	273	356	244	233	223
.9	213	204	195	187	179
1.0	173	166	160	154	148
1.1	143	138	132	128	124
1.2	120	116	112	108	105
1.3	102	99	96	93	91
1.4	88	86	83	81	79
1.5	77	75	73	71	69
1.6	67	66	64	62	61
1.7	59	58	57	55	54
1.8	53	52	50	49	48
1.9	47	46	45	44	44
2.0	43	42	41	40	39
2.1	39	38	37	36	36
2.2	35	35	34	33	33
2.3	32	33	31	30	30
2.4	29	29	28	28	28
2.5	27	27	26	26	25
2.6	25	25	24	24	24
2.7	23	23	22	22	22
2.8	22	21	21	21	20
2.9	20	20	19	19	19
3.0	19	18	18	18	18
3.1	17	17	17	17	17
3.2	16	16	16	16	16
3.3	15	15	15	15	15

Table A (continued)

		.02	.04	.06	.08
		431,500	107,875	47,944	26,968
3.4	14	14	14	14	14
3.5	14	13	13	13	13
3.6	13	13	13	12	12
3.7	12	12	12	12	12
3.8	11	11	11	11	11
3.9	11	11	11	11	10
4.0	10	10	10	10	10

TABLE B

PENETRATION RESISTANCE—PSI.

20-lb. Weight on Narrow Cone

		.02	.04	.06	.08
		863,000	215,750	95,888	53,936
.1	34,520	23,972	17,612	13,484	10,654
.2	8,630	7,132	5,992	5,106	4,400
.3	3,832	3,370	2,986	2,662	2,390
.4	2,156	1,956	1,782	1,630	1,498
.5	1,380	1,278	1,182	1,100	1,024
.6	958	896	842	792	746
.7	704	664	630	596	567
.8	538	512	488	466	445
.9	426	408	390	374	359
1.0	345	331	319	307	296
1.1	285	275	265	256	248
1.2	240	232	224	217	211
1.3	204	198	192	186	181
1.4	176	171	167	162	158
1.5	153	149	145	142	138
1.6	135	131	128	125	122
1.7	119	116	114	111	109
1.8	106	104	101	99	97
1.9	95	93	91	89	88
2.0	86	84	82	80	78
2.1	78	77	75	74	73
2.2	71	70	68	67	66
2.3	65	64	63	62	61
2.4	60	59	58	57	56
2.5	55	54	53	52	51
2.6	51	50	49	48	48
2.7	47	46	46	45	45
2.8	44	43	43	42	41
2.9	41	40	40	39	39
3.0	38	38	37	37	36
3.1	36	35	35	34	34
3.2	33	33	32	32	32
3.3	31	31	30	30	30
3.4	29	29	29	28	28
3.5	28	27	27	27	26
3.6	26	26	26	25	25
3.7	25	24	24	24	24
3.8	23	23	23	23	22
3.9	22	22	22	22	21
4.0	21	21	21	21	21

TABLE C

PENETRATION RESISTANCE—PSI.

40-lb. Weight on Narrow Cone

		.02	.04	.06	.08
		1,726,000	431,500	191,776	107,872
.1	69,040	47,944	35,224	26,968	21,308
.2	17,260	14,264	11,986	10,212	8,804
.3	7,668	6,741	5,972	5,325	4,780
.4	4,315	3,912	3,566	3,260	2,996
.5	2,760	2,553	2,364	2,201	2,052
.6	1,918	1,796	1,685	1,584	1,492
.7	1,408	1,331	1,260	1,195	1,132
.8	1,078	1,024	978	932	891
.9	852	815	782	750	719
1.0	690.4	654	638	620	592
1.1	570	550	531	513	496
1.2	479	464	449	435	421
1.3	408	396	384	373	363
1.4	352	342	332	324	315
1.5	307	299	291	283	276
1.6	273	263	256	250	244
1.7	238	233	228	223	218
1.8	213	208	204	200	195
1.9	191	187	183	179	176
2.0	173	169	166	163	160
2.1	157	154	151	148	145
2.2	143	141	138	135	132
2.3	130	128	126	124	122
2.4	120	118	116	114	112
2.5	110	108	107	105	104
2.6	102	101	99	98	96
2.7	95	93	92	91	89
2.8	88	87	86	84	83
2.9	82	81	80	79	78
3.0	77	76	75	74	73
3.1	72	71	70	69	68
3.2	67	66	66	65	64
3.3	63	62	61	61	60
3.4	59	59	58	57	57
3.5	56	56	55	54	53
3.6	53	52	52	51	50
3.7	50	49	49	48	48
3.8	47	47	46	46	45
3.9	45	44	44	44	43
4.0	43	42	42	41	41

TABLE D

PENETRATION RESISTANCE—PSI.

80-lb. Weight on Narrow Cone

		.02	.04	.06	.08
		3,452,000	863,000	383,552	215,750
.1	138,080	95,888	70,448	53,936	42,616
.2	34,520	28,528	23,972	20,424	17,612
.3	15,336	13,484	11,944	10,654	9,460
.4	8,630	7,824	7,132	6,520	5,992

Table D (continued)

		.02	.04	.06	.08
		3,452,000	863,000	383,552	215,750
.5	5,520	5,106	4,734	4,400	4,104
.6	3,832	3,592	3,370	3,168	2,986
.7	2,816	2,662	2,520	2,390	2,264
.8	2,158	2,052	1,956	1,867	1,782
.9	1,704	1,630	1,562	1,498	1,437
1.0	1,380	1,327	1,278	1,228	1,182
1.1	1,136	1,100	1,062	1,024	991
1.2	958	927	896	869	842
1.3	816	792	768	746	725
1.4	704	685	664	647	630
1.5	613	596	582	567	552
1.6	538	526	512	501	488
1.7	478	466	456	445	435
1.8	426	417	408	399	390
1.9	382	374	366	359	352
2.0	345	338	331	325	319
2.1	313	307	301	296	290
2.2	285	280	275	270	265
2.3	261	256	252	248	244
2.4	240	236	232	228	224
2.5	220	217	214	211	208
2.6	204	201	198	195	192
2.7	189	186	184	181	179
2.8	176	174	171	169	167
2.9	164	162	160	158	155
3.0	153	151	149	147	145
3.1	144	142	140	138	136
3.2	135	133	131	129	128
3.3	127	125	124	122	121
3.4	119	118	116	115	114
3.5	112	111	110	109	107
3.6	106	105	104	102	101
3.7	100	99	98	97	96
3.8	95	94	93	92	91
3.9	90	89	88	88	87
4.0	86	85	84	83	82

Discussion

CHARLES W. JOHNSON, Materials Engineer, and E. B. BAIL, Special Consultant, New Mexico State Highway Department—The cone device is admirably suited to the conditions existing in North Dakota, where great areas of fine-grained, relatively homogeneous soils exist. An impressive indication of soil uniformity is presented by Wise when he states that one subgrade test per mile is usually sufficient. On such soils it is practicable and structurally safe to design and build a flexible pavement of uniform thickness from end to end.

By contrast there is not, to our knowledge, any area in New Mexico where subgrade tests could be safely spaced more than 500 feet apart. Our soils are heterogeneous in the extreme; a relief map of this state shows a succession of mountain ranges separated by valleys in which are mixed all the products of erosion, and it is upon, and from this heterogeneous base that we must build our highways.

With a rather small field force at our disposal and a large mileage to test, the cone device is scarcely practicable for us. Furthermore, the method of averaging the bearing values at different depths, which, judging by the test results submitted by Wise,

does not, in the cases cited, require the reconciliation of vastly different values, would, in the case of our greatly differing underground composition, give rise to considerable doubt as to the validity of the average.

Wise presents an interesting correlation with other methods of test for determining thickness of flexible base, and refers to McLeod's paper published in Engineering News, June 9, 1949, Page 77. It is noted that he rates the 400 psi. cone bearing as indicating a pavement thickness of 9½ inches, and, further, that this thickness is satisfactory for a 9,000-lb. wheel load. He states that this corresponds to a CBR of 10.25 and to 24 blows for 6-inch penetration of the Housel Penetrometer.

The correlation of 400 psi. for the cone device and a CBR of 10.25, both indicating approximately 9½ inches of flexible base for 9,000-lb. wheel load is quite in agreement with the estimated thickness given by a method developed at the New Mexico Highway Testing Laboratory and presented at the 1947 meeting of the Highway Research Board. In this method the soil is compacted at a moisture content equal to the highest water content found under existing bituminous pavements in the same general area as the proposed construction. The CBR at 0.1 inch penetration is taken as the subgrade resistance value S. The resulting thickness d is taken as the thickness of granular foundation required to support a given wheel load. It is of interest to note from the curves shown on Page 101 of the 1947 Proceedings that a CBR of 10 requires 9 inches of base for a 9,150-lb. load.

By the use of the New Mexico Chart that we used on the Correlation Design of WASHO Materials, for a CBR of 10 we obtain a thickness requirement of 2 inches of asphaltic concrete plus 10½ inches of a good granular base or 3 inches of asphaltic concrete and 5½ inches of granular base.

By working backward from California's Thickness Design Chart III, starting with 9½ inches on Scale 1, and laying a scale through the center of Asphaltic Concrete on Scale H, Cohesiometer Value, a value of 14 inches is obtained on Scale G for the gravel equivalent. Now by laying the straightedge from 14 inches through the traffic index of 7, we obtain an R value requirement of 38. It would be interesting to know what the R value would be on typical subgrade soils having a psi. of 400 by the cone device.

We are much interested in the progressive decrease in bearing values reported by Wise. Data on this type are much needed and the North Dakota Highway Department is to be congratulated on initiating such a study. We would like to know if this decrease in bearing value is accompanied by a significant rise in moisture content. We would appreciate also information as to the gradation of the soil and the Atterberg limits.

Pavement Deflections and Fatigue Failures

F. N. HVEEM, Materials and Research Engineer
California Division of Highways

This is a continuation of the paper entitled "The Factors Underlying the Rational Design of Pavements" appearing in the 1948 Proceedings of the Highway Research Board. The original work indicated the importance of fatigue failures caused by resilience in the supporting soils. This paper describes the initial work of measuring deflections over a wide variety of pavements. Examples are shown illustrating the load-deflection curves where pavements are showing signs of failure and on other sections where conditions are good or excellent. In general, the deflections are directly proportional to load, although not in all cases. The deflections were measured under both single-axle and tandem-axle loads and the relationship between these two types of loading are established for several types of pavement.

Laboratory methods are discussed including the design of a resiliometer for measuring the resilient characteristics of soil samples and the design of a fatigue testing machine for measuring the relative flexibility of pavements. The study indicates that a comprehensive design procedure must provide a pavement structure that will either be capable of surviving the fatigue resulting from continuous flexing or have sufficient "stiffness" to reduce the flexing to an acceptable value.

● IN 1948, a paper was presented at the Twenty-Eighth Annual Meeting of the Highway Research Board wherein an attempt was made at identifying and classifying the numerous factors and properties of materials that singly or in combination affect the performance of highway pavements (1). A chart was used to analyze the relationship between the major and minor factors. While the original paper attempted to include all of the factors in the discussion under "Part 1, Analysis of the Pavement Design Problem," the solutions, test methods and design chart proposed at that time were aimed at providing answers to only two of the three primary basic problems shown in Figure 1.

In other words, the design procedures then proposed and which have since been followed in California and subsequently adopted by several other states and foreign countries are confined to anticipating the ultimate density, the amount of moisture which could ultimately be taken up by the soil, and the resistance value of the soil and base material when the worst conditions will have been reached. This procedure, however, does not provide safeguards against failure in the form of cracking or breaking up due to fatigue resulting from continual flexing or bending of the pavement structure under passing wheel loads, Figure 4. This third factor has long been recognized and appears as item three in the second column of Figure 1.

ANALYSIS OF THE PROBLEM

In order to simplify and further illustrate the relationships between the several factors and the structural adequacy of pavements, Figure 2 presents in tabular form the same three basic problems, together with the properties of pavements, bases, and soils which must be recognized and reconciled in order to provide an answer to each problem and thus produce a satisfactory pavement.

In Figure 2 the three problems are listed at the head of Columns 1, 2, and 3, while in the left-hand column the pertinent properties of the basement soils, bases, and pavements are listed in two separate groups. This arrangement is intended to indicate that the engineer must generally accept the basement soil with its inherent properties as it will exist in the roadbed. He must evaluate these properties by suitable tests in order to assign numerical values to the important variables.

It will be noted that the basement soil, whether in situ or imported, is considered to have four important properties which must be determined by separate tests and evaluated independently: (1) internal friction, R-value, measured by the stabilometer; (2)

cohesion, tensile resistance, measured by the cohesiometer; (3) swelling pressure, expansive force exerted during the absorption of water, measured by expansion pressure device; and (4) resilience, compression and rebound under passing loads, measured by resiliometer.

While some selection is often possible the engineer must generally accept the basement soil and deal with the properties as they will exist beneath the pavement after the passage of time, perhaps of several years. However, the engineer has greater freedom in choosing or providing the properties and dimensions of the pavement and the base materials. (The exercise of this choice is often called "engineering judgment.") The most-important of these properties is set forth in Figure 2 as (a) flexural strength, variously referred to as beam strength, slab strength and may be indicated by modulus

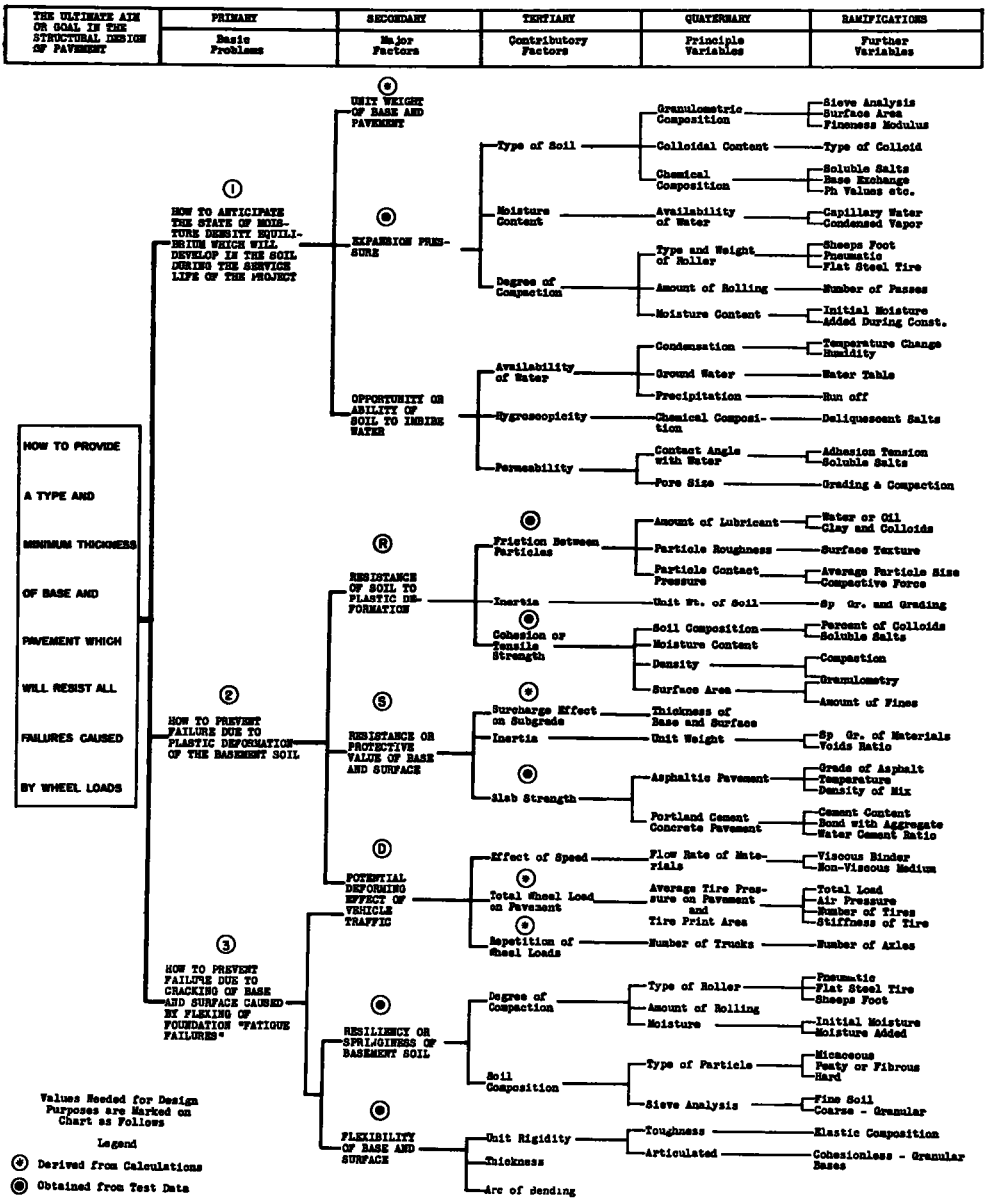


Figure 1. Analysis chart of factors affecting structural adequacy of pavements.

Properties of pavements and subgrades susceptible to measurement which must be taken into account or manipulated in the process of designing adequate pavements and bases		In order to select the type and thickness of pavement and base, three distinct problems must be considered and solved			
		1 EQUILIBRIUM	2 DISTORTION	3 REBOUND	
		How to maintain equilibrium of moisture and density in basement soil by restraining expansion	How to prevent plastic deformation of the basement soil under heavy wheel loads	How to prevent subgrade rebound from destroying the pavement through fatigue	
Properties	Test Method				
Basement Soil					Factors which must be evaluated to determine the problem
A. Internal Friction	Stabilometer		A. (Major)		
B. Cohesion	Cohesiometer		B. (Minor)	B. (Minor)	
C. Swell Pressure	Swell Dynamometer	C. (Major)	C		
D. Resilience	Resiliometer			D. (Major)	
Pavements & Bases					Factors which may be adjusted or manipulated in solving the problem
a. Flexural Strength	Cohesiometer Beam Tests		a	a	
b. Weight	Thickness Specific Gravity	b	b	b	
c. Flexibility or lack of Brittleness	Test Method being developed			c	

Figure 2. Relationship between fundamental factors governing structural design of pavements and bases.

of rupture values, tensile strength test or by the cohesiometer; this quality contributes to the "stiffness" of the pavement.

The second important contribution by the pavement and associated base layers is (b) direct weight resting on the subgrade, easily determined by computation knowing the proposed thickness and the unit weight of each layer. The "stiffness" factor of the pavement and base combination will also vary with thickness even of granular masses of low cohesive strength.

The third is: (c) flexibility (fatigue resistance), the ability to withstand repeated bending or flexing. The overall flexibility is influenced both by the thickness of the pavement section and by the elasticity. A new device is now under development in the laboratory to evaluate this property in terms of fatigue resistance.

In order to indicate how these various properties may affect the overall performance, in Figure 2 the index letters are transferred and arranged to indicate "answers" in the columns representing the three basic problems. Thus, it will be seen that Problem 1, "How to determine the ultimate moisture and density equilibrium," requires a knowledge of the potential expansion pressure of the soil as it will exist after construction operations; this tendency to expand can only be counteracted by placing a sufficient weight of cover (pavement plus base shown as b in Figure 2) over the soil to balance or oppose the expansion pressure. [1]

The second problem, which is to prevent failure from plastic distortion of the underlying material, requires that the internal friction be measured and, perhaps, the cohesive resistance also; although this latter can usually be neglected for design purposes. The expansion properties are significant here only because they may influence the amount of water taken up and thus indirectly affect the internal friction.

There are two solutions to this problem: (a) to use a pavement of high flexural

[1] Expansive tendencies can, of course, be reduced or eliminated by adding more water during or immediately following construction.

strength; (b) to place a sufficient weight of base and surface, or, as there are no weightless pavements, some combination of the two is employed.

As stated before, procedures for dealing with Problems 1 and 2 have already been set up and are being followed in a number of highway laboratories today. However, so far as is known there have been no organized attempts to deal effectively with Problem 3, which seems to be increasingly serious in recent years due to the great increase in the weight and numbers of heavy wheel loads on highway pavements. Therefore, this paper will deal primarily with pavement deflections under repeated load applications and the resultant fatigue failures.

Referring to the concept illustrated by Figure 2, it will be noted that Problem 3 adds increasing complexities. For Problem 3 it now seems that the internal friction or resistance value of the soil may not be directly significant and cohesive resistance probably plays only a minor part. The main consideration is the actual resilience[2] of the soil in the condition of moisture and density that will be characteristic of the materials after the pavement has been in place for some time.

The lower half of Column 3 indicates there are three possible answers or solutions to the third problem. If a pavement of sufficiently high slab strength is employed (a), then it will not be deflected beyond safe limits by the passing loads. Also, if a sufficient weight or thickness of stable granular material (b) is used in the base course, there will be no undue flexing of the pavement surface. Either or both types may have the required "stiffness."[3] And finally, at direct variance with the limited solutions for Problems 1 and 2, a thin flexible pavement (c) may serve quite well over resilient soils where a heavier, stronger, but more-rigid or brittle type will crack and perhaps show other signs of failure.

However, the materials engineer who must recommend an adequate overall design must make sure that whatever combination of pavement and base is proposed will adequately and simultaneously satisfy all three of the basic problems enumerated. In many cases thin pavements will not satisfy Problems 1 and 2. In view of the fact that methods dealing with Problems 1 and 2 were set forth in the 1948 paper (1) and have subsequently been improved, the following will be confined to a discussion of the factors that must be taken into account for a solution to Problem 3: How to prevent fatigue failures in the pavement due to flexing caused by alternate depression and rebound under moving wheel loads.

DEFLECTIONS

For the purposes of this discussion, the term deflection will be used in a limited and

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[2] "Resilience" is preferred to such terms as elasticity as we are here concerned with movements much greater than would be developed in many elastic solids such as glass, concrete, steel, etc. A new device, termed the "resiliometer," has been developed to measure this property of soils on laboratory specimens.

[3] The term "stiffness" has been borrowed from a report by L. W. Nijboer and C. van der Poel (2). Nijboer computes stiffness from the formula

$$S = \frac{F_p}{X_p} \quad (12)$$

for F_p equal to 10^4 N (1 ton) and 2×10^4 N (2 tons) respectively.

F_p = Force acting on pavement

X_p = Deflection of the pavement

Therefore, the term "stiffness" bears a simple mathematical relationship to the deflection of the pavement; as used by Nijboer, stiffness implies the resistance of all components including the pavement, bases, subbases and the underlying soil. For design purposes it seems preferable to us to associate the concept of stiffness with the pavement and base structures alone in which case there will not be a consistent relationship between stiffness and deflection as the character of the supporting soil will then represent a variable: "resilience."

special sense to indicate those movements of the pavement under traffic in the form of downward bending beneath the vehicle wheel followed by rebound after the load has passed on. For this purpose the term "deflection" applies to transient movements and is considered to be only one of several types of deformation which a pavement may undergo.

Figure 3 presents an outline of the terms together with contributory causes which are subdivided into a primary and secondary group. For the purpose of this discussion the following definition applies:

Deflection. A transient downward movement of the pavement when subjected to vehicle wheel loads. A deflected pavement rebounds shortly after the load is removed.

Pavement deflections have been a matter of interest to the writer for many years. While serving as a maintenance superintendent in 1924, prior to the use of bituminous

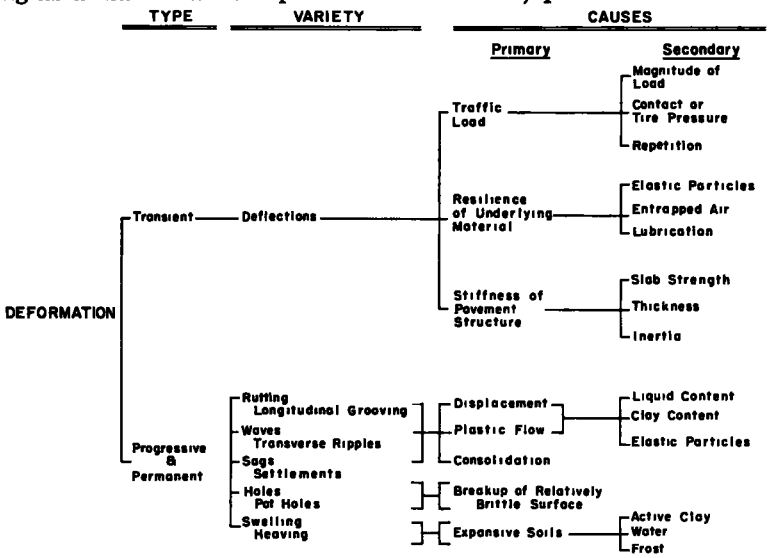

Figure 3. Analysis chart illustrating types of pavement deformation.

surfacings on California's rural highways, he observed that there were marked differences in the difficulty of maintaining untreated gravel roads; in many cases the behavior apparently bore some relationship to the character of the foundation soil. In other words, there were a number of examples on level grades where the gravel surfacing would remain in relatively good condition in cut sections where the roadbed consisted of solid rock as compared to shallow fill sections of fine-grained soils.

The troubles noted were chiefly in the form of raveling and potholing of the surface, but as the surfacing material was the same throughout, it seemed probable that there was a greater magnitude of flexing and bending under heavy wheel loads where the underlying soils were more resilient. Thirty years ago much-less attention was given to the selection of materials in the roadbed, and such material as leaf mold and vegetable matter was not always rigorously excluded as, we hope, is the case today.

In 1938, the laboratory of the California Division of Highways secured a General Electric travel gauge to measure deflections of pavements under rapidly moving wheel loads. This unit was used in scattered investigations throughout the years both on state highways and for test pavements constructed by the state and the Corps of Engineers. It was found nearly 15 years ago that asphaltic pavement deflections varied greatly with temperature. However, it was not until 1951 that an organized study was undertaken to determine the actual deflections that traffic was inflicting on California highway pavements. The data furnished herewith are intended to be a progress report of a study which is by no means completed. The data represent selected examples from 43 projects involving the installation of nearly 400 gauge units and over 2,500 individual gauge records.

In 1951, a newly constructed section of asphaltic-concrete pavement (less than 2 years old) was showing marked evidence of distress in the form of extensive cracking of the

type usually described as "chicken-wire" or "alligator" cracking (Figure 4). This section of road is a four-lane divided highway north of Los Angeles, on US 99 which is the principal truck route between Los Angeles and points north. The cracking was first observed and became most pronounced in the outer lane, which carries over 80 percent of all traffic and virtually all of the heavily loaded trucks. However, on a short stretch of this project all lanes of the pavement remained in good condition with no evidence of cracking.

Deflection gauges were installed in both cracked and uncracked areas, and the deflection of the surface was measured with reference to rods driven through the base and anchored in the underlying soil at various depths. Figure 5 illustrates a typical installation of these gauge units. Figure 6 is a plot of the deflections which were measured on this project against reference rods 3 feet long, using axle loads ranging from 11,000 to 29,000 lb. as indicated by the abscissa scale.

It will be observed that there is a marked difference in the magnitude of the deflections measured where the pavement is badly cracked compared to the area where there is no evidence of cracking.

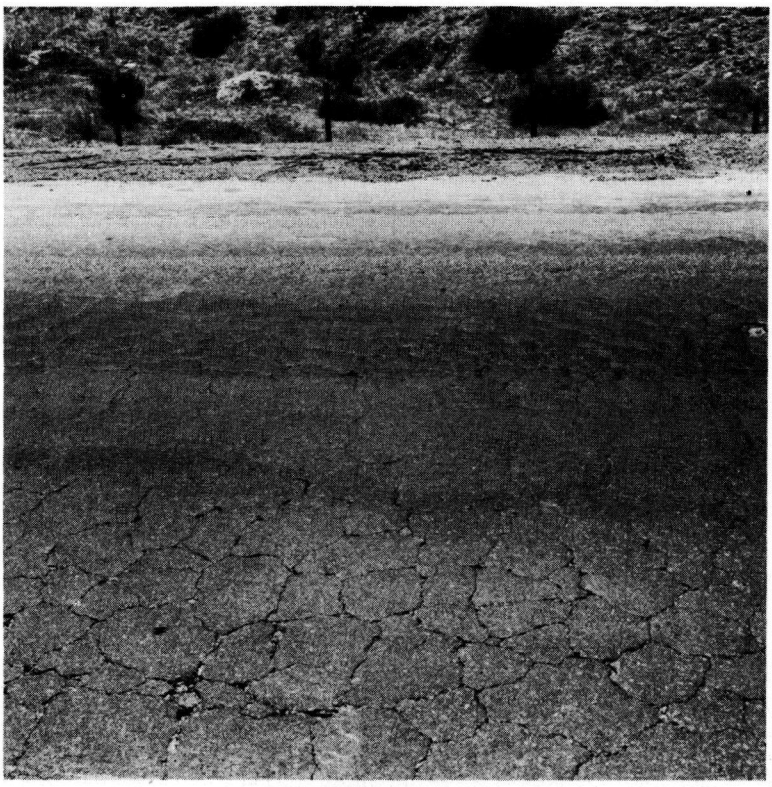

Figure 4. Typical illustration of "chicken wire" or "alligator" cracking.

Most of the deflection measurements that have been made to date indicate that in general the measured deflection bears a linear relationship to the load applied, although this relationship does not everywhere hold true. As will be shown later, certain types of soil (especially where deflections are high) develop a distinctive load-deflection pattern that is not in accord with the linear relationship indicated in Figure 6. In any event, it will be apparent that the badly cracked portions of the pavement have been continuously subjected to much higher deflections than has the section where no cracking is in evidence. It might be argued, of course, that the deflections are higher because the pavement has cracked and thus lost continuity and slab strength. Undoubtedly, the deflections are greater when the slab continuity has been destroyed; however, the de-

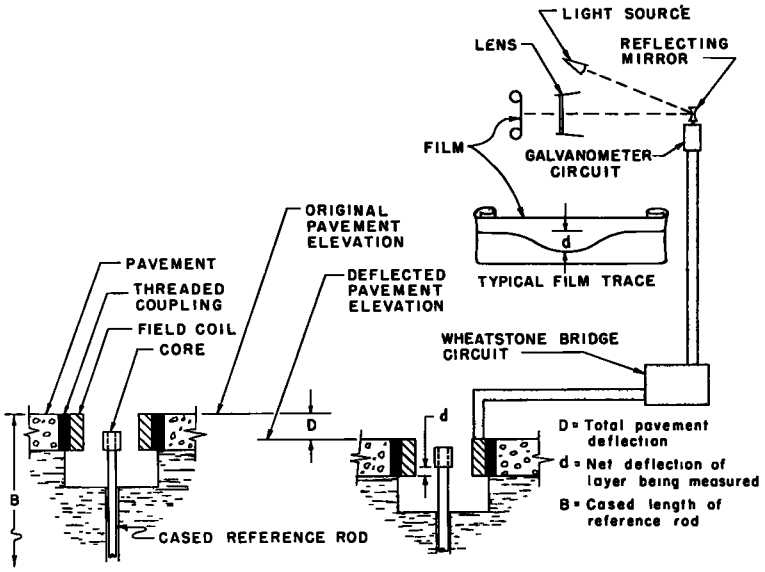

Figure 5. Schematic diagram of apparatus for deflection measurements.

flection in the passing or inner lane at the same location is also relatively high compared to most uncracked pavements, and the absence of cracking in the inner lane can only be attributed to the fact that it carries relatively few heavy loads.

The structural section used on this project is shown in Figure 7. It will be noted that it is as heavy and substantial as that used on the New Jersey Turnpike, for comparison. The asphaltic-concrete surface has a stabilometer value of 45. The crushed granite base has an R-value of 77 and a CBR of 161, the subbase has an R-value of 74 and a CBR of 125. However, it now appears that the basement soils on this project are definitely resilient, and there is also evidence that the asphalt has become hardened; therefore, the pavement is more brittle than desirable.

The original asphalt had a penetration between 120 and 150; however, recoveries of the asphalt made in 1951 by the Abson method show an average penetration of 36 after one year on the road. The records indicate that plant temperatures were quite moderate and unusually well controlled ranging between 275 and 285 F. While subjected to heavy

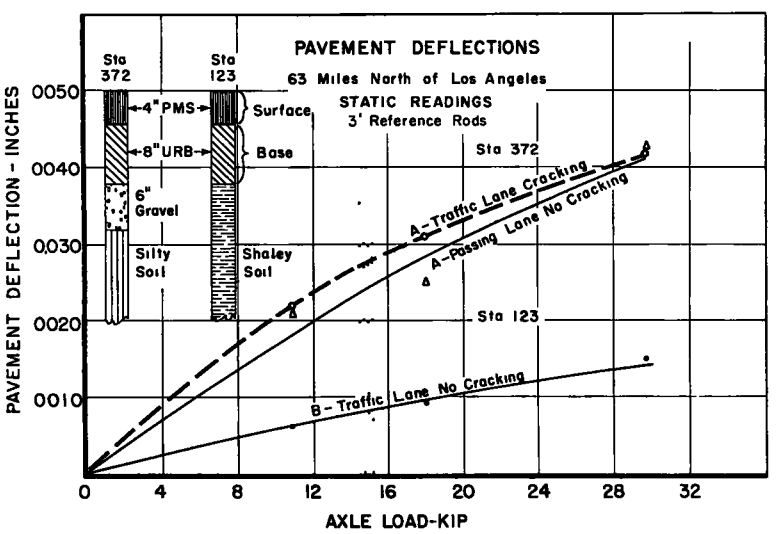

Figure 6.

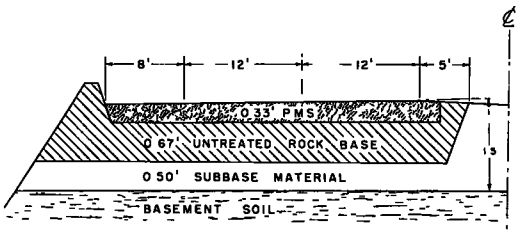

Figure 7. Typical cross section road VII-LA-4-J, Station 372+20 pavement cracked.

traffic and badly cracked, as noted above and shown by Figure 4, this pavement has remained smooth and undistorted, indicating that the so-called failures were entirely due to flexing and bending.

The second project that was investigated is on the Bayshore highway, US 101, one of the heaviest-traveled routes in the state, being the main traffic artery from San Francisco to the south. A portion of this highway was reconstructed on new alignment in 1947, traversing an area of mud flats which required imported embankment materials up to 25 feet in depth surfaced with 5 inches of asphaltic concrete resting on 8 inches of crushed stone over 24 inches of sand subbase.

Failures in the asphaltic surfacing in many areas on this section (Figure 8) became evident within 1 or 2 years after construction. As an investigation conducted by the laboratory could discover no deficiencies in the quality of the asphaltic pavement or of the base material, it was decided to measure the magnitude of deflections. Figures 9, 10, 11 and 12 illustrate the deflection measurements made on this project using reference rods of different lengths.

Figures 9 and 10 represent the deflections at two locations where the pavement is in good condition.[4] Figures 11 and 12 represent readings taken at points where cracking and distress of the asphalt pavement were evident. Figure 13 gives a profile view of the measured deflections illustrating the length of pavement involved in these deflection zones.

Attention is drawn to the evidence that truck loads can affect the pavement foundation to a depth of 18 feet or more. It will be obvious, of course, that the magnitude of vertical deflection is not of itself completely significant, as the tendency to break or rupture the pavement will depend primarily upon the sharpness of the arc or curvature of the pavement surface.

Engineers in Sweden have concluded that when the pavement is bent in an "arc" having a radius less than 100 feet, failures would result. It may be that asphalt pavements in Sweden are more flexible; in any event we have been unable to arrive at a similar value for California conditions, although the general idea seems sound.

While the vertical deflection measured with reference to a rod 18 feet in depth is of the greater magnitude, it may well be that the most-severe stresses in the pavement are associated with compression and rebound in the upper layers of the embankment, which may produce sharper bending and consequent greater stress in the pavement slab. In other words, the shape of the depressed area will undoubtedly vary with variations in both basement soil and pavement structure.

It must also be recognized that a moving wheel load causes a sharp reversal of stress from tension to compression in every portion of the pavement in the wheel path. There is evidence to indicate that in many cases the sharpest bend and consequently the greatest stress is outside the wheel contact area.

After the initial studies outlined above, it was tentatively assumed that the compressibility and rebound in the top 8 or 9 feet of the roadbed soil would be of greater significance and more readily correlated with performance than the total overall deflection.

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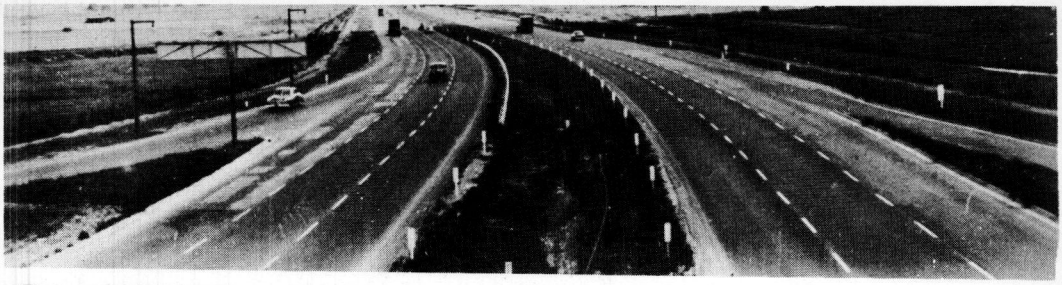
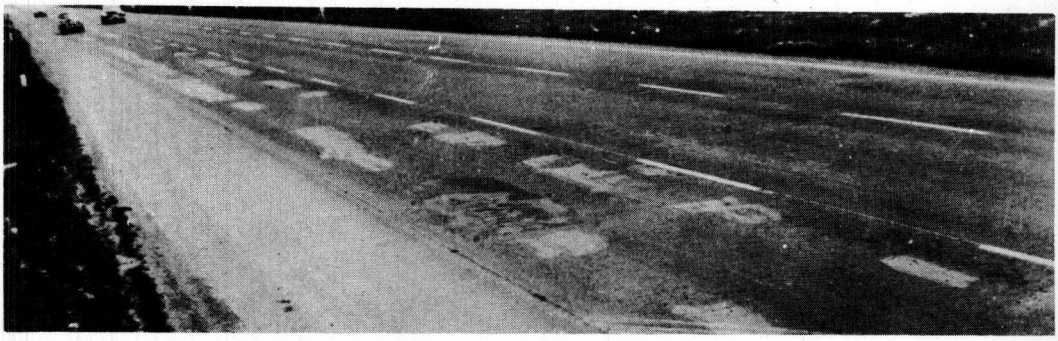
[4] One of these was in an area where vertical sand drains had been placed in the embankment. This poses an interesting question about the function and performance of this currently controversial type of installation. There is no evidence that the fills have settled more rapidly with the sand drains than adjacent areas without this provision. Borings alongside the sand columns brought no evidence of dewatering of the soil. Nevertheless the pavement is in better condition and as stated above the deflections are markedly lower. Perhaps the 3-foot layer of pervious sand placed as a blanket over the vertical drains is the answer.



Sta. 395± General Outside Lane Failure



Sta. 498± General Outside Lane Failure

Figure 8. Typical failed areas. San Francisco Bayshore Freeway.

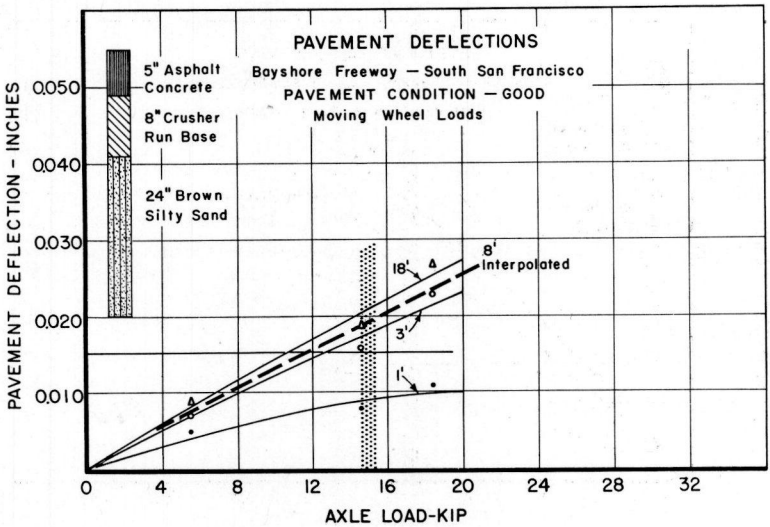


Figure 9.

Therefore, on the more-recent work, these depths have been used for purposes of comparison, although it is readily admitted that the question of how best to employ a simple measurement of deflection as an index of destructive bending movements has yet to be settled in our minds. However, in order to "start somewhere" we have compared deflections referred to an 8-or 9-foot reference rod under 15,000-lb. single-axle loads. This means that what we are actually reporting is the compression and rebound of the soil in the upper 8 or 9 feet of the roadbed.

An asphaltic-concrete pavement on US 40 between Sacramento and San Francisco has given an excellent performance since 1937. Deflection measurements were made as shown in Figure 14 on a section that was definitely in good condition. Here it will be noted that the compression and rebound in the top 9 feet of the roadbed under a slow-moving 15,000-lb. axle load is 0.012 inch. In this instance we have also shown, for comparison, the deflection caused by a static or standing load. While the difference here is greater than most, it is true that deflections under static loads are always greater than under moving loads.

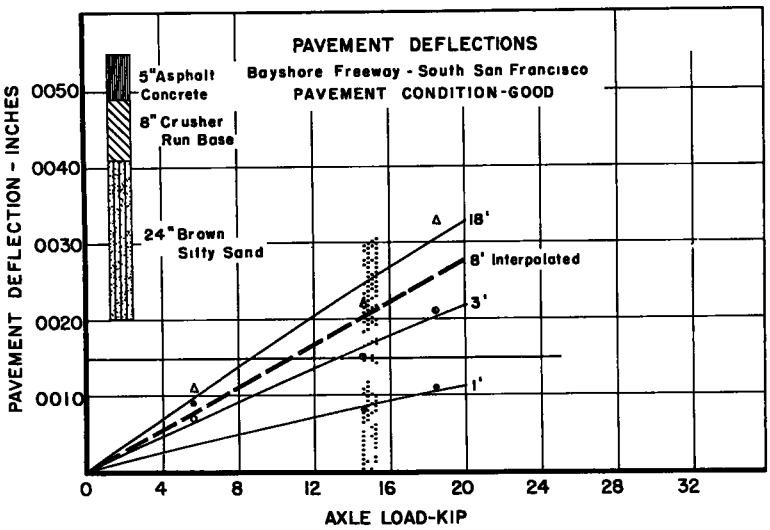


Figure 10.

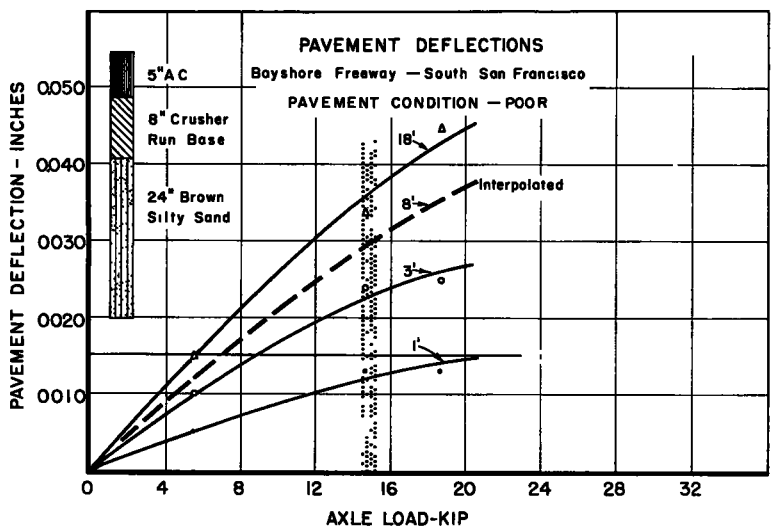


Figure 11.

By way of comparison, Figure 15 refers to a portland-cement-concrete pavement 5 miles north of Eureka. This pavement is some 28 years old and in fair condition, considering its age, type of foundation, and amount of traffic. Here the deflection related to the upper 9 feet is moderate, being 0.016 inch.

Next, a few asphaltic surfaces were studied, on the Redwood Highway where the road is subjected to heavy log hauling. One section across swampy ground (Beatrice Flats)

has undergone periodic settlements throughout the years. As a result, the roadbed has been built up by additional layers of gravel and bituminous surfacing, until at the present time the total thickness of gravel is over 30 inches. The asphalt surface is now in excellent condition and has remained so for some time. Again one may note from Figure 16 that the deflection with reference to a 9-foot rod is 0.017 inch.

An even-more-striking example is the present road on the Redwood Highway (south of the town of Scotia) which was reconstructed in 1946 using a 3-inch plant-mix surface over an 8-inch cement-treated base supported by a substantial subbase of pit-run gravel. Here the deflections under a 15,000-lb. load for a 9-foot depth are only 0.009 inch (Figure 17). The excellent appearance of this surface and freedom from maintenance cost over a period of 8 years testify to the fact that this pavement is adequate for the heavy traffic which it must sustain.

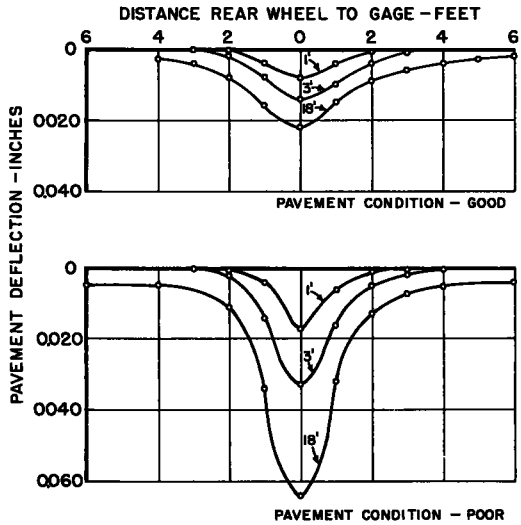


Figure 13. Pavement deflections, Bayshore Freeway, South San Francisco, 14,000 lb.-wheel load.

In marked contrast are the high deflections measured on a section of old secondary road 21 miles southeast of Eureka, where the old Warrenite pavement shows evidence of extensive cracking. This section has been resurfaced several times, and the maintenance costs have been high. Figure 18 illustrates deflections as high as 0.140 inch.

One might comment at this point that damage to a pavement is not necessarily in direct proportion to the magnitude of the deflection. Once the pavement is cracked into small blocks, it then acquires the ability to bend, and this "articulated" structure can accommodate considerable movement without necessarily progressing rapidly to complete failure. Such cracked pavements are a worry to the engineer; however, in many cases they will carry traffic for a long time, proving that true flexibility is a desirable characteristic.

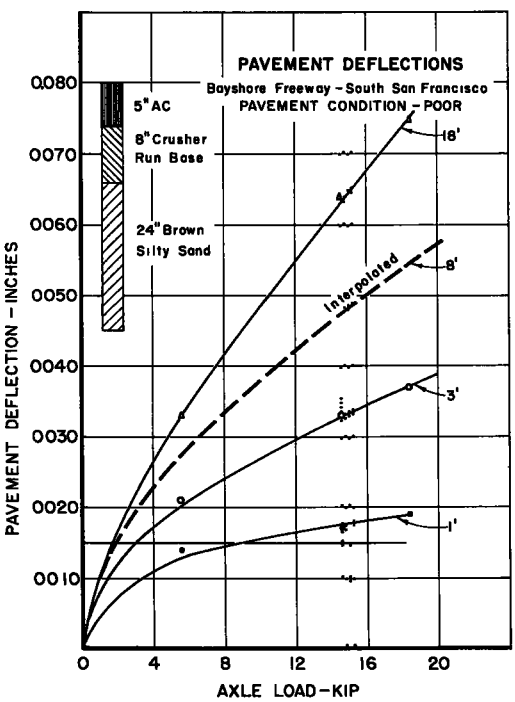


Figure 12.

Figure 19 shows a comparison between the deflections measured in a pavement supported in one area by a cement-treated base and in another by a gravel base. The gages were set in the outer wheel track of a traveled way, supported by 6 inches of cement-treated base. Gages were also placed in the adjacent shoulder section, surfaced with plant-mix resting upon 19 inches of pit-run sand and gravel containing appreciable amounts of clay and silt. Here it will be noted that the cement-treated base apparently has a marked effect in reducing the pavement deflections.

Additional confirmation of the effect of slab strength is shown by Figure 20, which illustrates the low deflections of a section where 4½ inches of plant-mixed surfacing is supported by 8 inches of cement-treated

base. This section is about 43 miles north of Los Angeles on the main highway known as the Castaic Bypass. The appearance of this pavement is excellent, with only an occasional transverse shrinkage crack. The use of cement-treated bases appears to be an effective means of reducing the deflections in many cases.

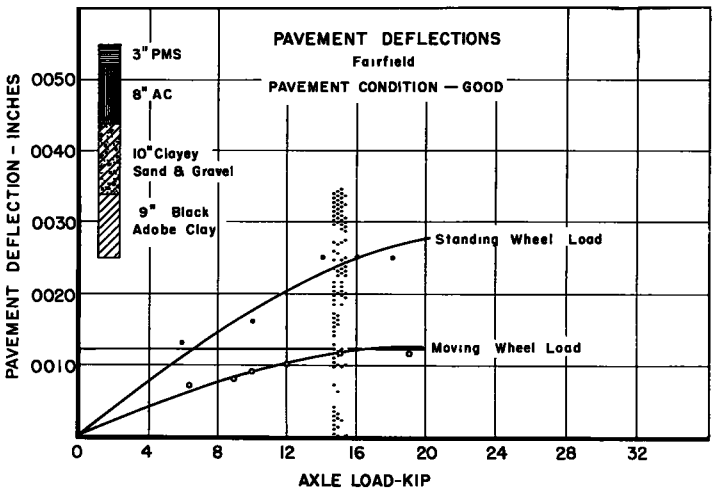


Figure 14.

However, there is also evidence that deflections may be reduced to equally acceptable limits by means of heavy gravel or crushed-stone bases, as is illustrated by Figure 16 and Figure 21. Figure 21 shows the same comparison as Figure 19, but in this case the difference in deflection between the pavement resting upon a cement-treated base and the adjacent shoulder supported only by sand and gravel is small. The excellent condition of both shoulder and traveled way are in accord with the low deflections measured.

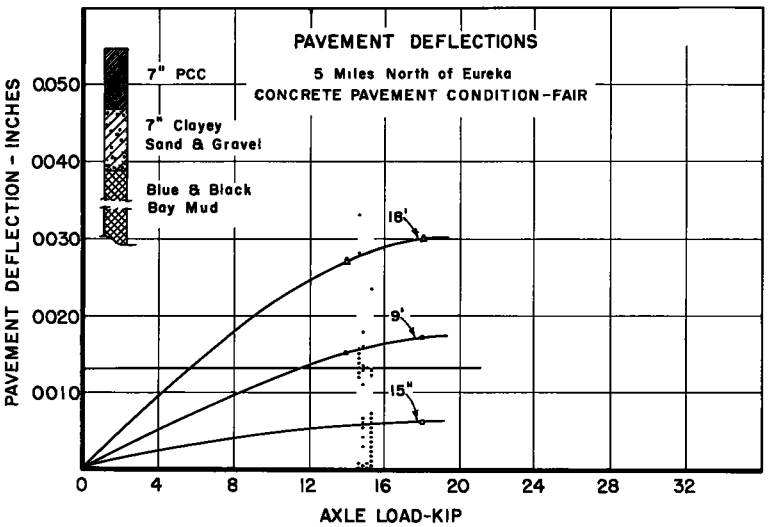


Figure 15.

A number of deflection measurements have also been made on concrete pavements, most of which are in relatively good condition, however. Some of these curves showing deflection versus load are arranged in Figure 22. It is a difficult matter to make comparisons between the deflections of a concrete pavement and those of a bituminous type.

The deflections will vary throughout the length of the average concrete slab, which at night or in the early morning is usually curled up at each end, thus losing contact with the subgrade. The deflections, therefore, are generally greater at the ends of the slabs than at the midpoint, and this curling or warping is affected by both temperature and moisture. Therefore, in order to measure deflections which reflect the bending of the slab due to compression and rebound of the subgrade, it is necessary to place the gages in the slab midway between the joints. In any event, it will be noted that the deflections are all comparatively low for concrete pavements in good-to-excellent condition.

It must be pointed out, however, that the ends of most concrete slabs are being continually flexed under passing vehicles, not because of subgrade compression but because the ends are often unsupported for a distance of 5 to 7 feet from the joint. Therefore, failure and breakup of concrete pavements may, in many cases, be unrelated to subgrade compression and rebound. Obviously, the problem of measuring and evaluating pavement deflections refuses to remain simple.

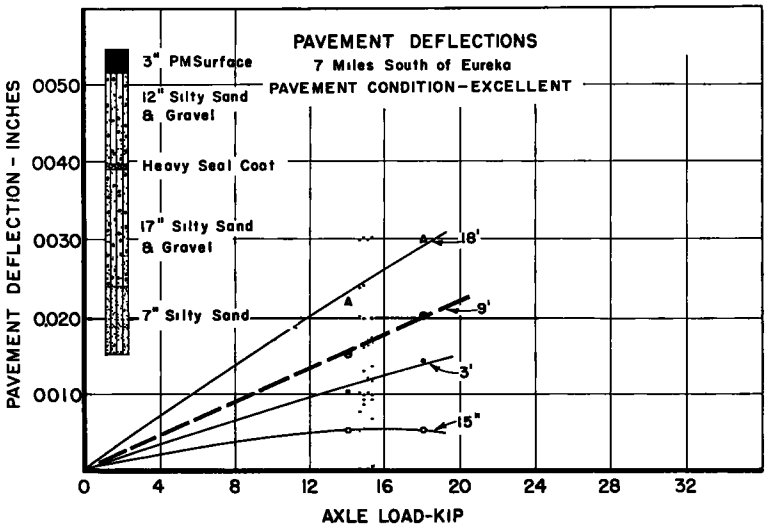


Figure 16.

As an aid in visualizing the shape of the depression "basin" in the pavement, a three-dimensional model was carefully constructed to an exaggerated scale. Figure 23 is a photograph of this model, representing a typical deflection pattern of a bituminous pavement on a gravel base under a 9,000-lb. dual-tire wheel load.

PAVEMENT CONDITION VERSUS DEFLECTIONS

A few examples were found where pavements undergoing fairly high deflections appeared to be in good condition, as indicated by some of the solid lines in Figure 25, but it will, of course, be obvious that asphaltic pavements inevitably vary somewhat in their ability to withstand repeated flexing without evidence of cracking, (Figure 32). There are differences in the aggregate gradations, in the grade and amount of asphalt. There are differences in ages of the pavements, in the thickness and differences in the climatic temperature range.

As mentioned above, most of the deflections shown thus far have indicated an approximately linear relationship with load; that is, the magnitude of the deflection is in direct proportion to the magnitude of the load. However, in one area of the state (on the coastal highway near San Luis Obispo) marked cracking of the pavement has been observed on two separate contracts separated by a few miles but utilizing similar materials in the imported subbase layers.

A plot of all deflections measured on these sections shows a characteristic curved pattern, (concave downward) indicating that the deflections are disproportionately high for the lighter loads. Nevertheless, it is true here, as elsewhere, that the measured

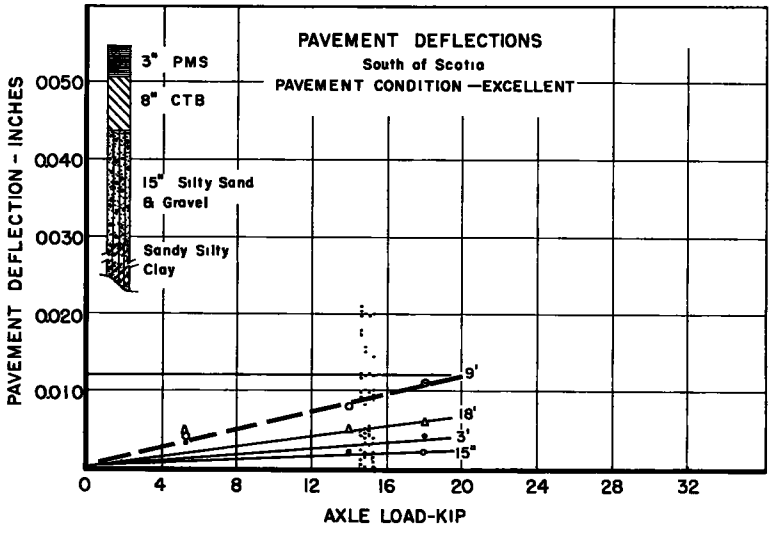


Figure 17.

deflections under 15,000-lb. axle loads in areas where the pavement is in good condition are generally less than 0.020 inch. Deflections made in cracked and failed areas on these sections generally exceed 0.025 inch. Figure 24 shows deflection measurements on this section.

The data shown herewith represent only selected examples of a large number of readings that have been made over California pavements. Figure 25 is a summary chart showing a comparison between the deflections characteristic of cracked pavements compared to those found where the surfacing is in good or excellent condition.

Deflection measurements have been made by the Corps of Engineers on airport pavements and by the Bureau of Public Roads and the Highway Research Board on the experimental test tracks of Road Test One-MD in Maryland(3) and on the WASHO track in southern Idaho (4). There has not been an opportunity or time to compare all of the available deflection data in order to establish general laws or rules. Knowing something of the variations which may exist in asphaltic paving mixtures, it would be unwise to make too-positive statements at this time concerning the amount of deflection which an individual pavement in a given area and climatic environment may safely withstand. However, considering that the study is incomplete and that further evidence may cast a different light on these conclusions, it now seems clear that the type of dense-graded asphaltic pavements frequently constructed in California (approximately 3 inches thick) will not long endure repeated flexing that exceeds 0.020 inch; heavier pavements appear to be limited to even lower values.

It must be pointed out that this study does not undertake to say exactly how much deflection is being produced by the

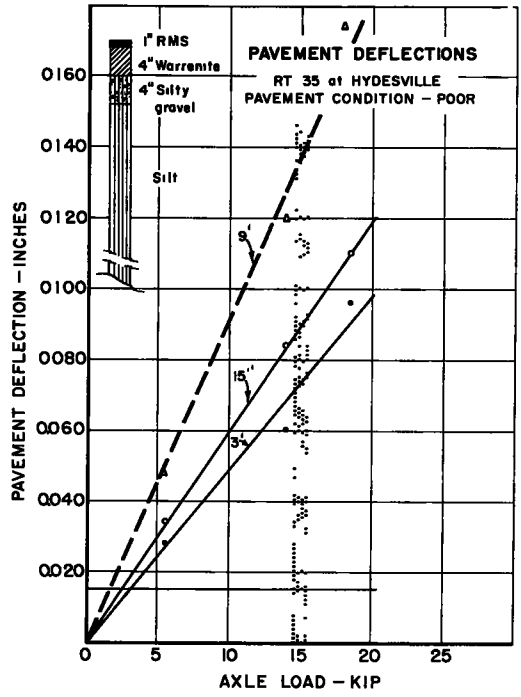


Figure 18.

current truck traffic passing over the road. As the failures are a fatigue phenomena, cracking is the result of both the magnitude of bending, or flexing, and the number of repetitions.

Most highway traffic represents a distribution or range of loads; while a moderate number may reach or even exceed the 18,000-lb.-axle-load limit, it is evident that the "average" wheel load must be somewhat less. Therefore, we have a complex problem in evaluating traffic where a few heavy loads cause high deflections and the many lighter loads cause lower but more-numerous bending repetitions.

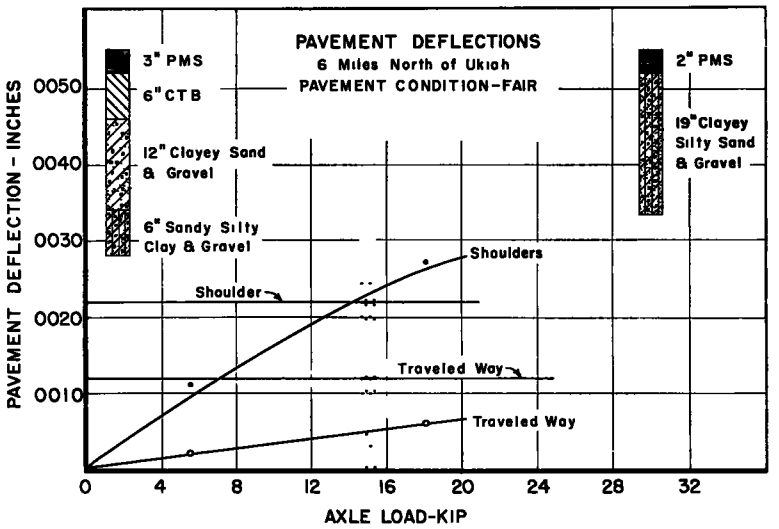


Figure 19.

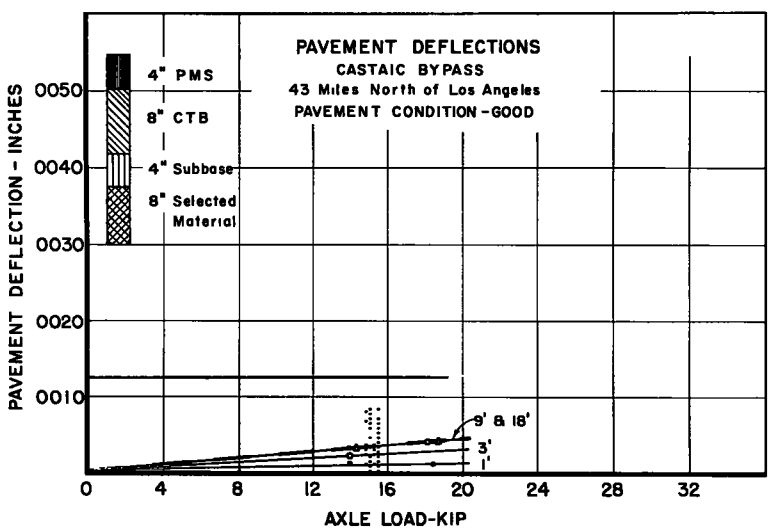


Figure 20.

We have assumed that an overall summation would be equivalent to an equal number of repetitions of axle loads of about 15,000 lb. On this assumption the cracking and fatigue failures of most pavements are attributable to a large number of deflections having an effective equivalent greater than 0.020 or 0.025 inch. Thinner or more-flexible pavements would obviously raise this limit of tolerance; or in the absence of heavy loads, the pavement would not be subjected to the magnitude or number of bending stresses.

VIBRATIONAL METHODS VERSUS DEFLECTIONS UNDER SLOW MOVING LOADS

Thus far we have discussed pavement deflections produced under slow-moving truck loads and the apparent relationship between these deflections and the condition of the pavement surface. The study has many interesting ramifications that have not been touched upon. For example, we were able to undertake some comparisons between the deflections caused by slow-moving loads on pneumatic tires and those developed by vibrational means.

Through the courtesy of the Shell Oil Company, the vibration tester developed in Holland (2) was made available, and measurements of strain, deflections, and velocity of wave propagation were taken during July 1954 at a number of locations on California highways where electronic gages had been previously installed and earlier readings secured. Attempts to establish a correlation between the deflections under wheel loads and those produced by the vibration machine were not too successful, probably for several reasons: (1) the lapse of time between the GE gage readings and the measurements with the vibration machine and (2) the fact that the vibrator operates through a heavy superimposed dead load, which probably tends to suppress some of the amplitude of movement.

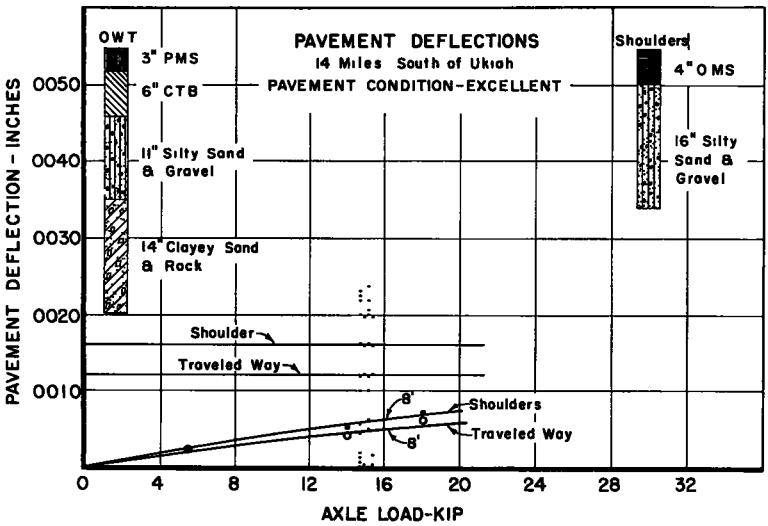


Figure 21.

Some comparison was made between deflections indicated by the Shell vibrator and those obtained by the Benkelman beam (4) on the same day. While it is apparent that no close correlation exists, there is a general relationship, even though the range of values obtained by the Benkelman beam under a 9,000-lb. wheel load is greater than the dynamic deflections registered by the vibration machine developing a force equal to 2 metric tons. Figure 26 shows a Benkelman beam being used to measure deflections caused by a heavy wheel load.

In general, it appears that there is no major difference in the evaluation of pavement stiffness which would be arrived at by either of the two methods, and the low cost, simplicity and speed of operation with the Benkelman beam device makes it an attractive instrument for the initial study of pavement deflections. The Benkelman beam does, have the limitation that it is impossible to identify the layer of material beneath the pavement that is responsible for such deflection as may occur. For this purpose we have found no substitute for the electric units, which make it possible to install a series of reference rods of varying lengths and thus identify the layer or horizon beneath the pavement that is chiefly responsible for the compression and rebound action. The GE gages also avoid a possible error that might result with the Benkelman beam where the length of pavement depressed is greater than 8 feet (Figure 13, for example).

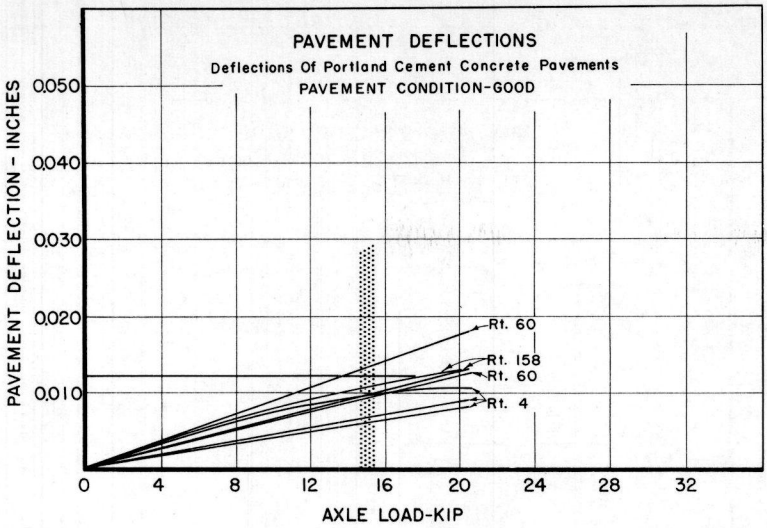


Figure 22.

Work in this laboratory has not progressed far enough to permit setting up a laboratory procedure giving information which will enable the designer to anticipate conditions of resiliency in the basement soil. Work on these lines is under way and we now believe we are justified in feeling optimistic about the outcome.

NEW TESTS AND DESIGN PROCEDURES

It appears that there are three major subdivisions of the laboratory work and the analytical steps necessary to develop a solution for this problem: First, is the measurement of deflections which are characteristic of existing pavements. This investigation requires many measurements over as wide a variety of pavement types and conditions as possible. From this study it should be possible to establish the magnitude of deflection which is characteristic of the failed sections compared to the amount shown by pavements in good condition. This work is under way in California and some of the preliminary results have been discussed and illustrated in the first part of this report.

In order to utilize the findings in the daily work of highway and airport engineers and to develop more realistic designs, it will be necessary to have laboratory means for evaluating the potential resilience of soils and proposed foundation materials and to be of any use for the average highway program such tests must be performed

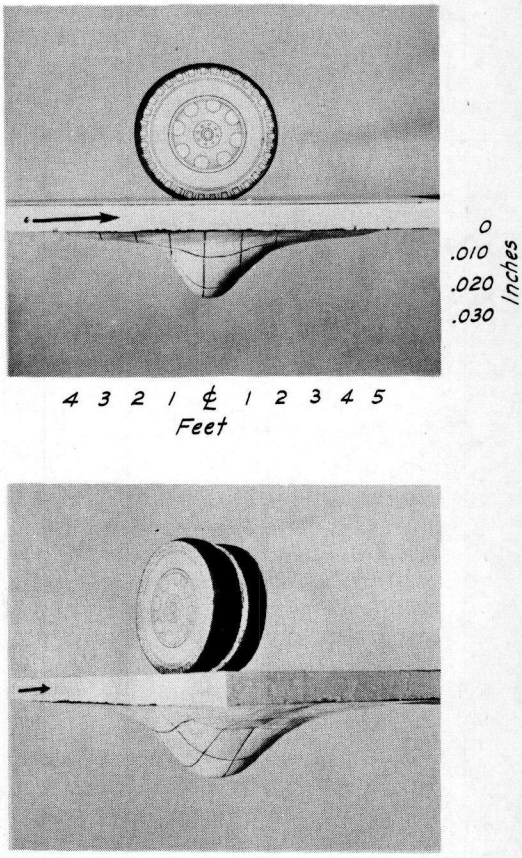


Figure 23. Model showing deflection. Pattern under a dual wheel load.

on samples taken in advance of design and, of course, even further in advance of actual construction.

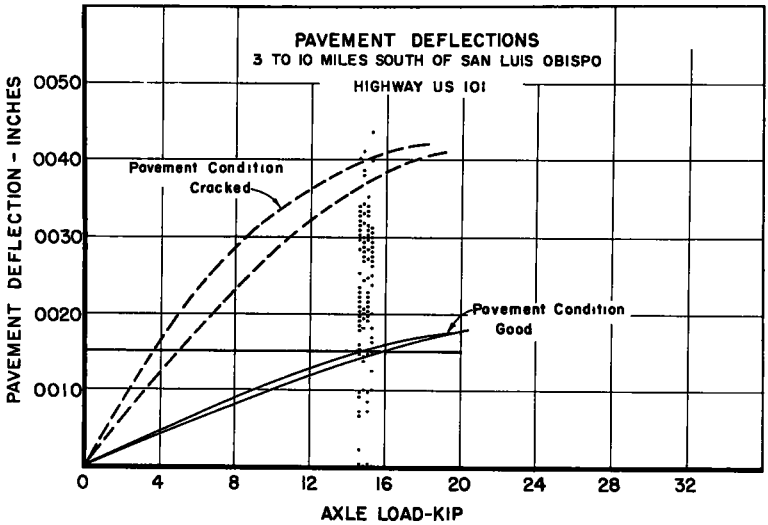


Figure 24.

MEASURING RESILIENCE OF SOILS

The first model of a resiliometer was developed and constructed in this laboratory in 1946. Preliminary trials indicated that it was possible to measure differences in the compression and rebound characteristics of soils, but work on the device was side-tracked for a time due to pressure of other projects. An improved model was constructed in 1954, and work was well under way until interrupted by a fire in the laboratory in March 1954. Since that time, resiliometer Model 3 has been designed and constructed (Figures 27 and 28). For the first time results appear to be consistent, and it now seems that it will be possible to measure and evaluate potential resilience using the small standard stabilometer specimens 4 inches in diameter and 2½ inches in height.

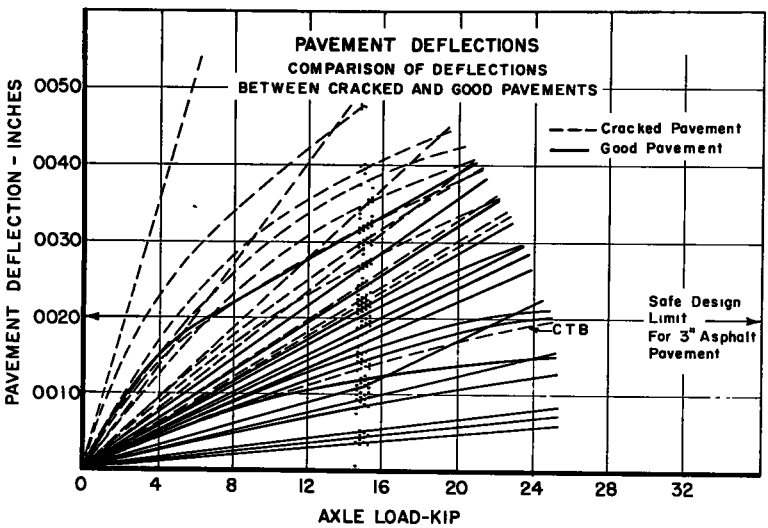


Figure 25.

Attempts to develop a satisfactory laboratory device and technique are only well started, and it would be premature to make positive statements or attempt final con-

clusions at this time. For example, it is not yet clear what pressures should be used in the resiliometer cycle in order to subject specimens to forces of the same order of magnitude as the pressures transmitted to the basement soils through the overlying layers of base and pavement.

Obviously, of course, in the actual roadbed the pressures will vary with depth; but for practical routine testing purposes, it would be much-more convenient to deal with a single figure for resilience using a single standard pressure in the test apparatus.

Tentatively, therefore, we think that a pressure of 20 psi. may be about right. Figures 29 and 30 show readings obtained with the resiliometer. Resiliometer readings are in terms of the volume of displacement or compression and have not yet been correlated with linear units of pavement depression.

Figure 29 is an expansive resilient soil from southern Idaho. This graph illustrates vividly that resilient properties are not manifest until the voids in the soil are filled with water, after which the susceptibility to compression and rebound increases rapidly with further increase in the moisture content. It is easily demonstrated, of course, that an expansive soil will take up moisture well beyond the point usually referred to as "maximum density"and "optimum moisture content."

Figure 30 illustrates resiliometer measurements on samples of well-graded gravel. Here the addition of moisture tends to diminish even the small amount of resiliency that exists.

While several details of technique and laboratory procedure are still to be settled, these results seem to warrant the belief that a successful test procedure can be evolved. It will be observed that the magnitude of deflection and rebound increases with increasing moisture content, after a certain value has been exceeded, and also increases with increasing unit pressure. These preliminary results strongly suggest that the flexing of pavements under passing wheel loads may undergo a sharp increase in magnitude as soon as the subgrade moisture reaches the saturation point. This is especially true of the agricultural soil types containing appreciable amounts of fine materials or clay and probably entrapped air.

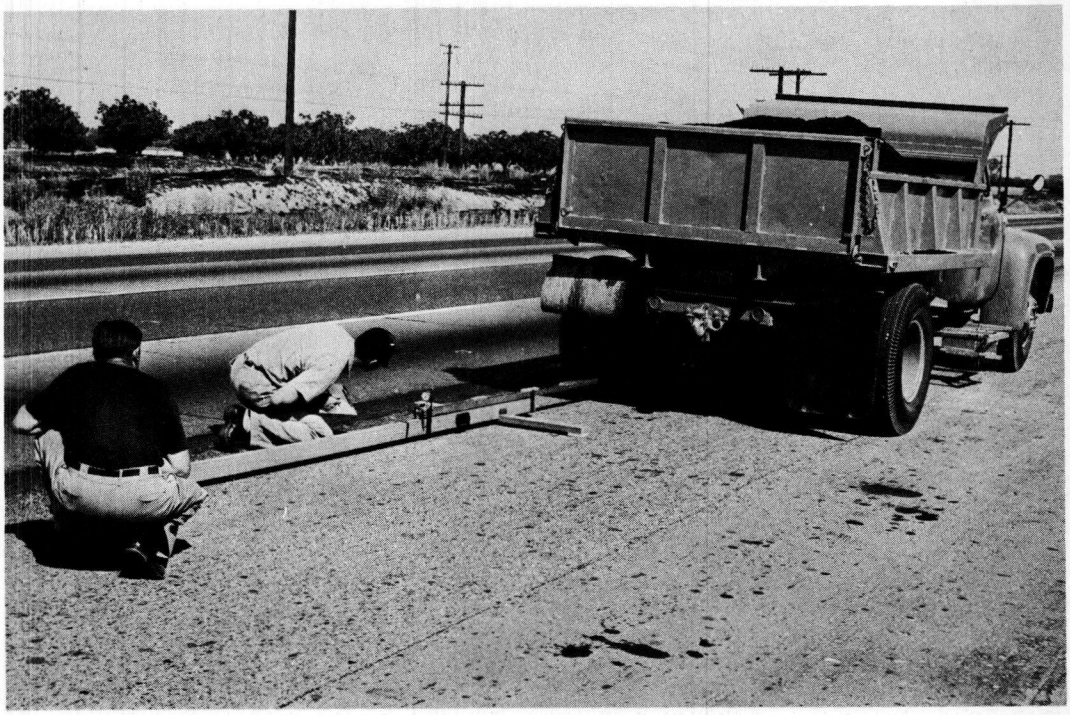


Figure 26. Benkelman beam being used.

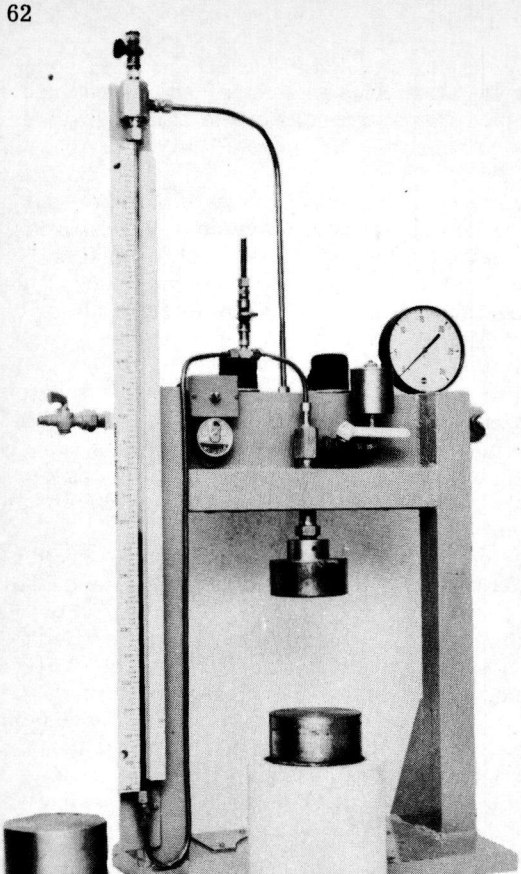


Figure 27. Resiliometer.

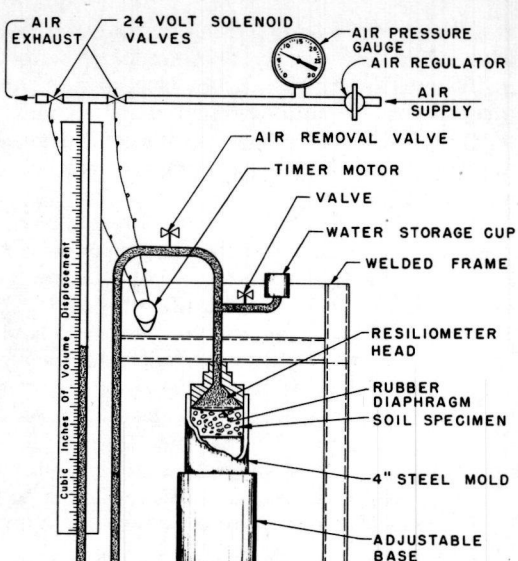


Figure 28. Resiliometer, schematic diagram.

As granular materials (such as clean sands, gravels, or crusher-run bases) characteristically show very-low values in the resiliometer, it is beginning to appear that there may be a closer correlation and parallelism between results in the stabilometer and measurements in the resiliometer than was first expected. Before resiliometer results were available, the idea was entertained that, when the soil pores were filled with water, the combina-

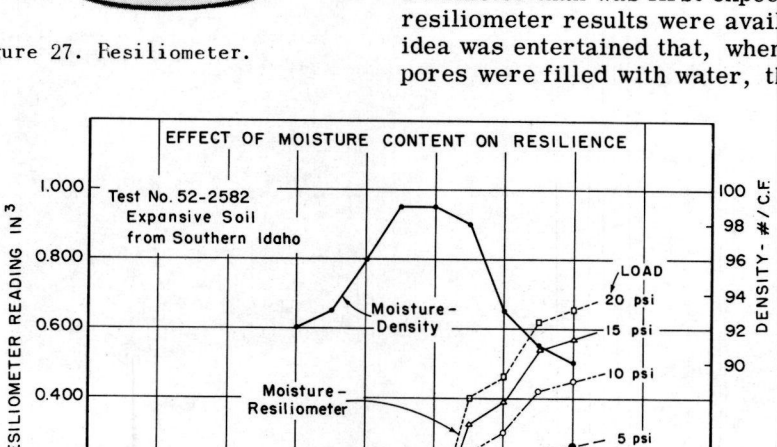


Figure 29.

tion would be virtually incompressible and, consequently, the resilience ought to be very low. Actual tests, however, have shown that if the soils do display any appreciable

resilience the range of movement increases with increasing moisture content beyond the point where the voids are first filled with water. Obviously, any granular mass cannot contain more water than the void space will accommodate. But while this void space is comparatively stable and fixed for a clean sand or gravel, such is not the case in fine-grained soils of the expansive type. Here the capacity for moisture will increase markedly as the soil expands. On such soils the internal resistance (R-value) will diminish,

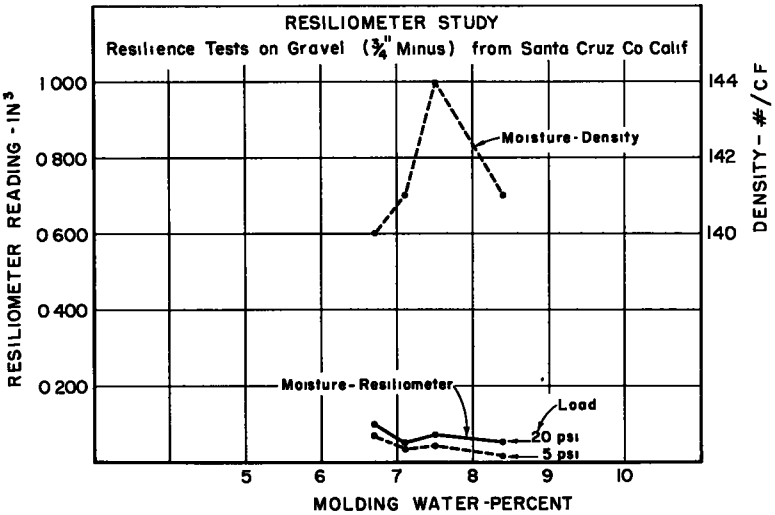


Figure 30.

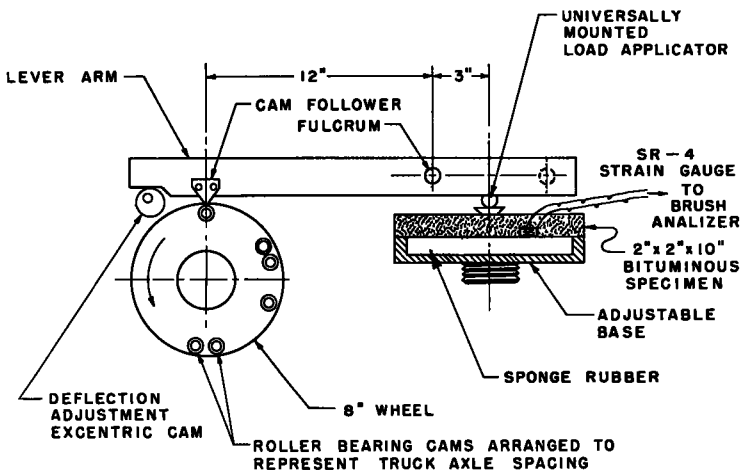


Figure 31. Bituminous pavement fatigue tester.

but it appears that the springiness or resilience will increase.

While the foregoing trends seem fairly evident, a great deal of work is yet to be carried out before the resiliometer becomes a proven device for the routine testing and evaluation of the resilient properties of soils.

FATIGUE RESISTANCE OF ASPHALTIC PAVEMENT

A second evaluation which must be made in the process of rational design concerns the ability of various types of pavement to withstand continued bending and flexing under the repeated action of traffic. How flexible is a "flexible pavement"? This character-

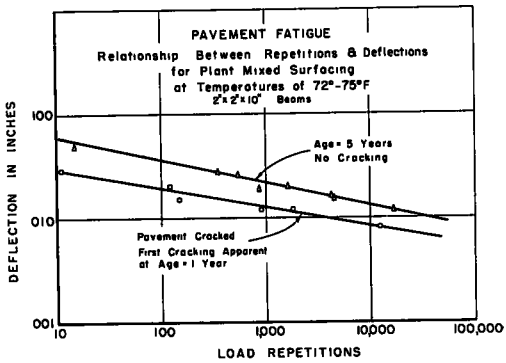


Figure 32.

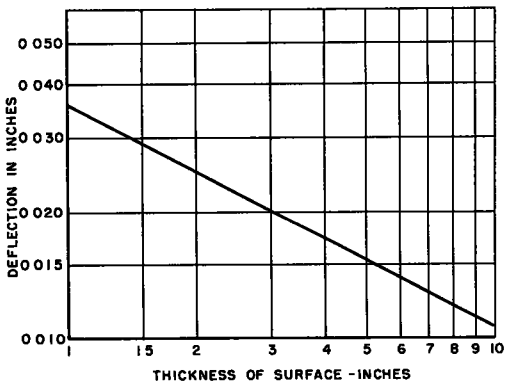


Figure 33. Permissible deflection under 10 million equivalent wheel loads.

istic will be more difficult to evaluate individually by test of pavement samples performed in advance of actual construction. In the first place, it will be difficult, if not next to impossible, to manufacture laboratory specimens that will have all of the properties of aged asphalt pavements that have been under traffic for some years. At the present time it seems questionable whether satisfactory and truly representative specimens can be formed in the laboratory for the purpose of measuring pavement flexibility. Fortunately, it does not appear that such a procedure will be absolutely necessary, as it should be possible to establish characteristic limiting values typical of pavements which have been in use for several years.

It is well known that asphalts tend to harden with the passage of time; therefore, asphalt pavements are undergoing constant change in their properties because of oxidation, loss of volatiles, polymerization and increasing density under the action of traffic. It seems that any evaluation of the ability of an asphalt pavement to withstand fatigue failures must be based on observations of actual performance on the road. Characteristic safe values can be set up for design purposes as is common practice for all structural materials. However, it should be possible to confirm evi-

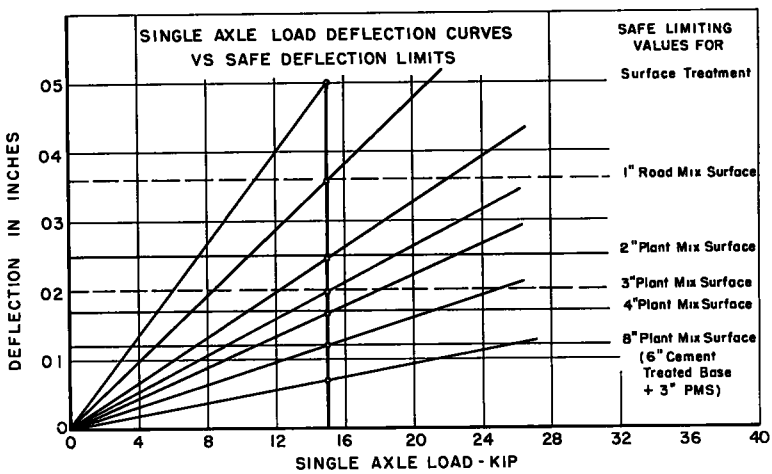


Figure 34.

dences of performance by the use of beams or test specimens sawed from actual pavements.

Reports of work at the University of Illinois on the fatigue of partland-cement concrete have indicated that a sufficient number of repetitions of a load that equals only 50 percent of the modulus of rupture value will ultimately cause failure (5). The modulus of rupture value is not easily or accurately determinable on a ductile, yielding material such as a specimen of asphalt pavement. However, studies have been made on the fatigue

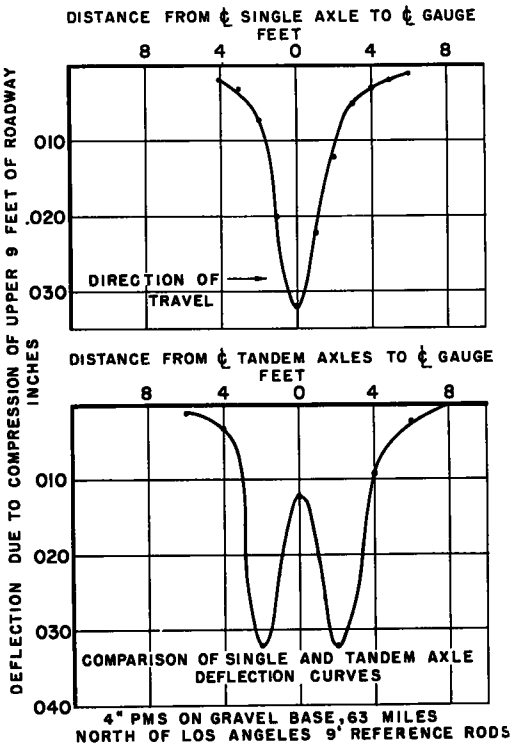


Figure 35.

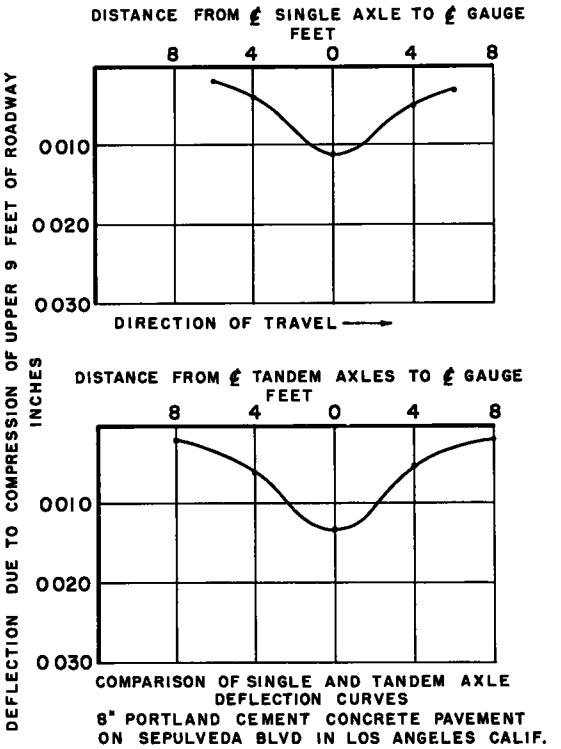


Figure 37.

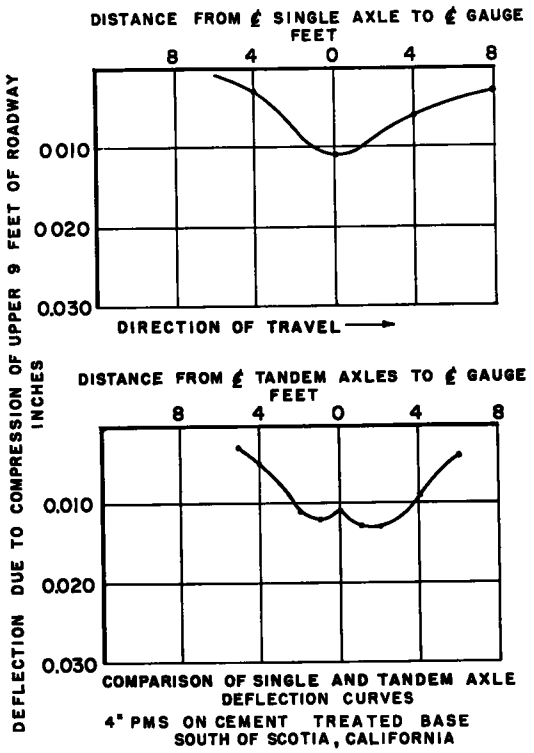


Figure 36.

resistance of small asphaltic pavement beams in the University of Washington under the direction of R. G. Hennes (6). Also, recent reports from Sweden indicate an interesting relationship between tensile

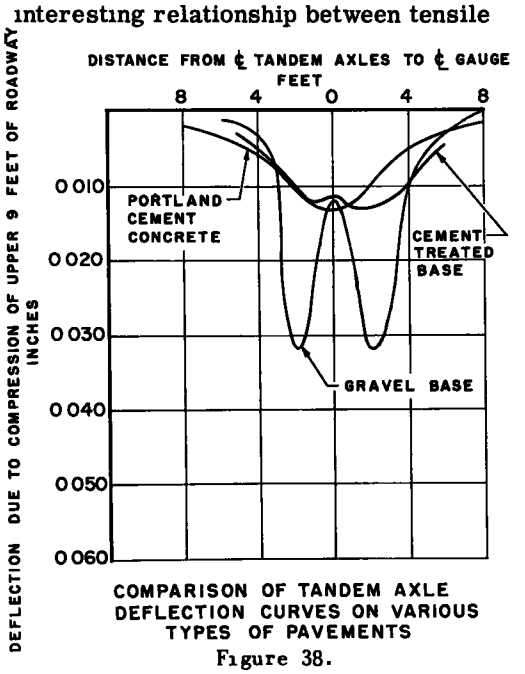


Figure 38.

strength and temperature (7).

Studies are now under way in this laboratory with a newly constructed device for measuring the fatigue resistance of typical asphalt pavements (Figure 31). Only a few results are available at the time of writing this paper, and the device for measuring fatigue or flexibility of bituminous pavements will undoubtedly undergo some changes and improvements, chief of which will be means for maintaining accurate temperature control. However, the initial trial results are interesting, and Figure 32 illustrates the results obtained on small beams of asphaltic pavement cut from slabs taken from a road surface. These preliminary results are unexpectedly uniform and show a definite relationship between the magnitude of deflection and the number of repetitions required to produce failure.

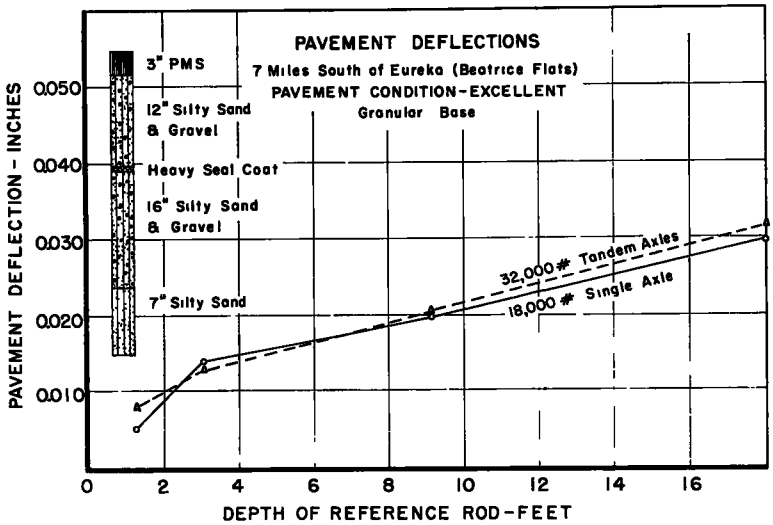


Figure 39.

It will take some time to establish the relationship between deflections of these small beams and the deflection of a pavement slab under heavy wheel loads, but there is little room for doubt that such a relationship does exist. Curve A in Figure 32 represents an uncracked pavement. Curve B represents beams from a cracked pavement. It is evident that the quality of the pavement is one of the variables.

In connection with the work of the WASHO Road Test, William Carey, of the Highway Research Board, experimented with a device to apply sudden rapid loads to beams of an asphaltic pavement. He suggested that we might do some work along the same lines and also suggested the use of a soniscope. Soniscope tests have been performed in our laboratory with some interesting results tending to confirm the work reported from Sweden (7); however, we believed that it would be necessary to subject pavement specimens to repeated loading in order to simulate actual road conditions. Hence, the device for measuring fatigue susceptibility in asphalt pavements was constructed with a cam arrangement operating at speeds to simulate the sequence of wheels on a multiple-axle truck.

In the preliminary trials, small beams 2 by 2 by 10 inches cut from asphaltic pavements have failed by cracking after being deflected 0.008 inch repeated 12,000 times at a temperature of 75 F. to 78 F. While this magnitude of deflection on a short beam can not, as yet, be compared directly to the deflections measured on the roadway (Figure 13), there can be little doubt that such pavements would fail after fewer repetitions when temperatures are lowered to the range typical of winter conditions throughout most of the United States.

RATIONALIZATION OF THE DATA

In the light of the foregoing, it may be visualized that, in addition to the design chart suggested in 1948 (1), a second design process will need to be established to provide a

sufficient depth or strength of pavement which will reduce the deflection to a value which the pavement can successfully tolerate throughout its entire economic life or to find means for constructing a flexible pavement that will not be damaged by the magnitude of bending stresses involved.

It appears that we can now make a start in suggesting tentative values for a safe scale of permissible deflections for current pavement types. It is obvious, of course, that the overall flexibility of any engineering material will vary with the thickness of the slab or beam, other things being equal.

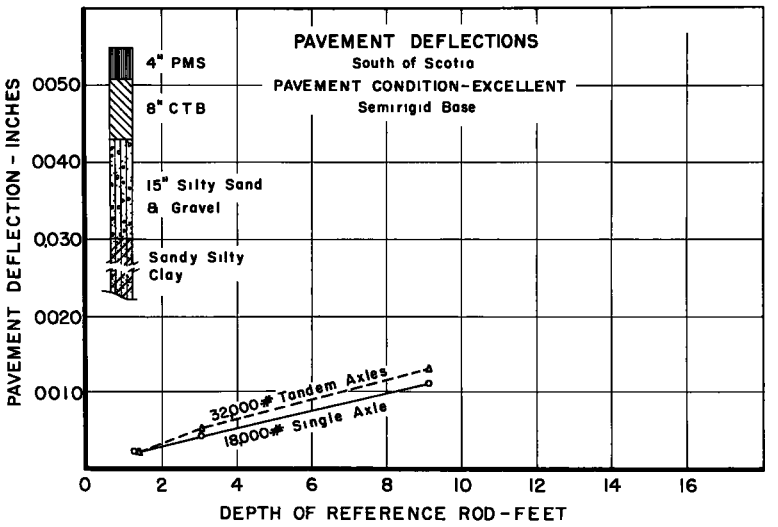


Figure 40.

Observations of these deflections and pavement performance seem to justify the suspicion that any superior flexibility of present day asphaltic pavements may be due largely to their generally thinner sections and, of course, varies with the amount of asphalt and age. The data seem to raise the question: Are present day asphalt pavements ultimately any more flexible than concrete pavements if constructed to the same thickness and if compared at low temperatures? Subject to many exceptions and individual variations, however, Table 1 appears to be a reasonable approximation of values for safe maximum deflections for several types of pavement and base construction.

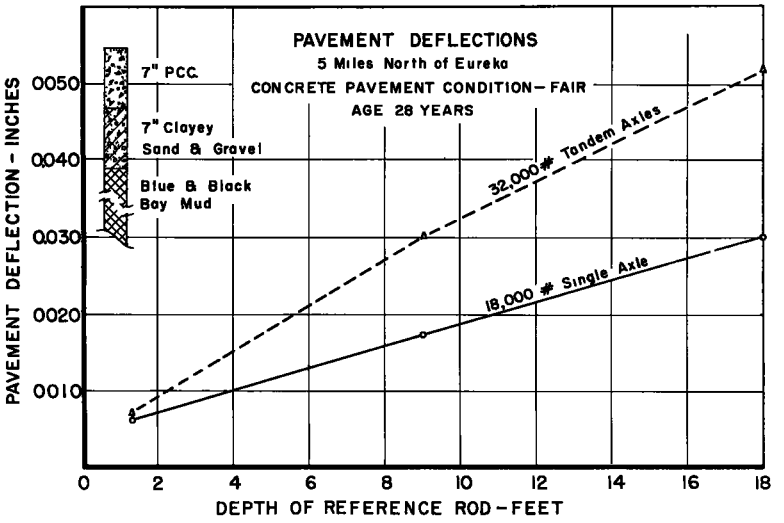


Figure 41.

TABLE 1

Thickness of Pavement	Type of Pavement	Max. Permissible Deflection for Design Purposes (Tentative)
8-in.	Portland Cement Concrete	0.012-in.
6-in.	Cement Treated Base (Surfaced with Bituminous Pavement)	0.012-in.
4-in.	Asphalt Concrete	0.017-in.
3-in.	Plant Mix on Gravel Base	0.020-in.
2-in.	Plant Mix on Gravel Base	0.025-in.
1-in.	Road Mix on Gravel Base	0.036-in.
½-in.	Surface Treatment	0.050-in.

(Bear in mind that the thickness of pavement indicated may or may not be adequate to satisfy the demands of problems one and two as outlined previously.)

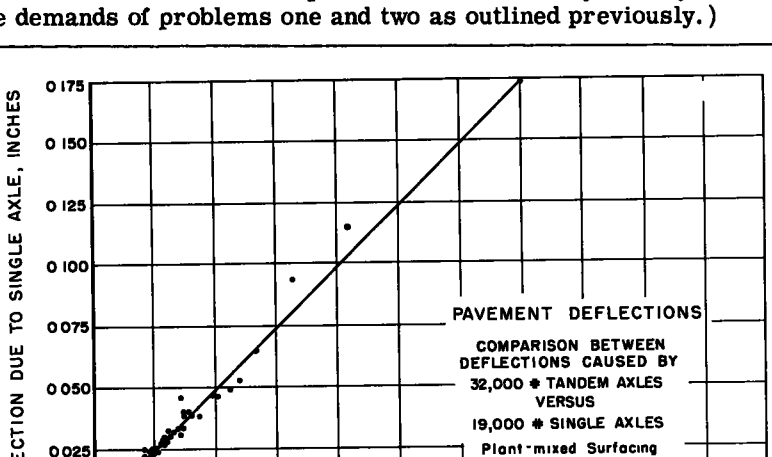


Figure 42.

The tentative deflection values of Table 1 are shown as a curve (Figure 34). The curve intended to indicate the safe limits of deflection under some millions of repetitions by heavy wheel loads. It is here assumed that a mean value would range somewhere in the neighborhood of 15,000-lb. single-axle loads. Many instances are known, of course, where the heavy traffic is almost entirely represented by trucks hauling logs, gravel, or similar commodities where all axle loads will be near to (or frequently exceed) the legal limits. Figure 33 shows a suggested straight-line relationship on a logarithmic grid between pavement thickness and permissible deflection.

In view of the fact that the majority of the deflections indicate a linear relationship between deflection and magnitude of the axle or wheel loads, a chart may be constructed, (Figure 34) showing the relative deflections which would be developed under a range of loads, the lines being drawn through points above the 15,000-lb. axle load corresponding to the various maximum deflections suggested in Table 1 and Figure 33.

A relationship suggested by W. N. Carey and A. C. Benkelman may prove to be more-consistently significant than simple deflection measurements alone. This relationship is expressed as

$$b = \frac{d}{a}$$

where b = bending index
d = deflection in inches

a = one half the axis of the load deflection area; that is, the distance in inches from the center of tire contact to the edge of the deflected area.

TANDEM AXLES VERSUS SINGLE AXLES

Among interesting facts brought to light by these field measurements of deflections is the evidence of a variable, but apparently orderly, relationship between the deflections resulting from single-axle loads compared to those caused by loads placed in the close proximity that occurs with tandem axles.

In the majority of pavements studied, the deflection measurements have been recorded for both single-axle and tendem-axle loads. Figure 35 shows the rapid reversal of pavement bending under a 31,000-lb. tandem axle on a section of badly cracked asphaltic pavement illustrated in Figure 6 and, for comparison, the pattern registered by a single-axle 18,000-lb. load. Figure 36 shows the deflections under both a single axle and tandem axles on the excellent pavement supported by a cement-treated base shown previously by Figure 17. Figure 37 illustrates the deflections of a concrete pavement under a single-axle load and the deflections of the same slab under tandem axles.

For bituminous pavements on gravel bases the foregoing indicates that when two axle loads are closely spaced, as in the case of tandem axles, then each trip of a truck produces a repetition of load for each axle regardless of spacing. For concrete pavement, on the other hand, there is little or no rebound between such closely spaced axles; therefore, a tandem axle should be counted as one axle load for purposes of summarizing the effects of traffic in equivalent wheel load (EWL) computations (see Figure 38 for comparisons, for instance).

The differences between the effects of single-axle and tandem-axle loads are further evident when the total amount of pavement deflection is compared. Figure 39 illustrates that an 18,000-lb. single axle and a 32,000-lb. tandem axle produce almost exactly the same deflections for depths up to 9 feet. This is the same pavement referred to by Figure 16.

This same close correspondence has been noted on most of the pavements consisting of a bituminous surface over a granular base. Figure 40 shows that where a cement-treated base exists there is a small, but consistent difference, the semirigid base showing greater deflection under a tandem representing two 16,000-lb. axle loads than for a single 18,000-lb. Finally, Figure 41 shows the marked difference where a concrete pavement is involved. Here the deflection under a 32,000-lb. tandem axle is much greater than for the 18,000-lb. single axle.

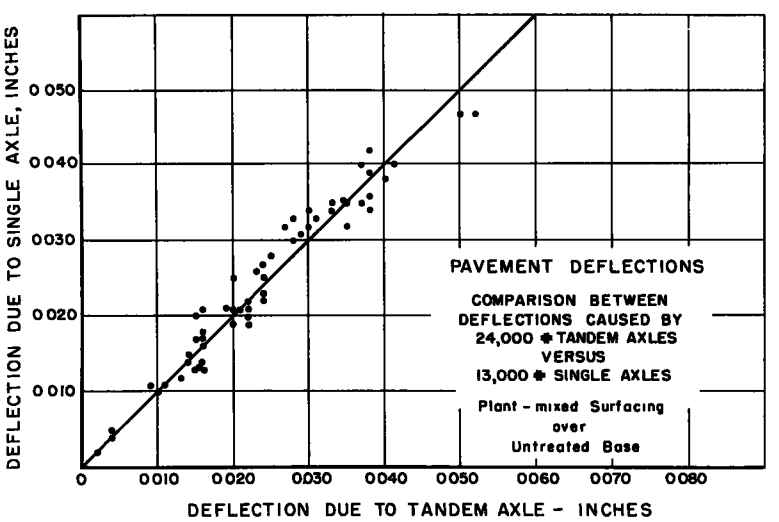


Figure 43.

Thus, it appears that the deflections of bituminous pavements over a gravel base subjected to a 32,000-lb. load on tandem axles show an average value almost identical to that produced by a 19,000-lb. single axle. Figure 42 shows the close and consistent relationship between all the available deflection values on bituminous pavements over gravel bases for both tandem and single axles at these loads. Similarly, Figure 43 shows that a 24,000-lb. tandem is equal to a 13,000 single axle. However, the relationship over a cement-treated base is indicated in Figure 44 as 32,000-lb. tandem equals 21,000-lb. single. For portland-cement concrete (Figure 45) 32,000-lb. tandem equals 24,000-lb. single.

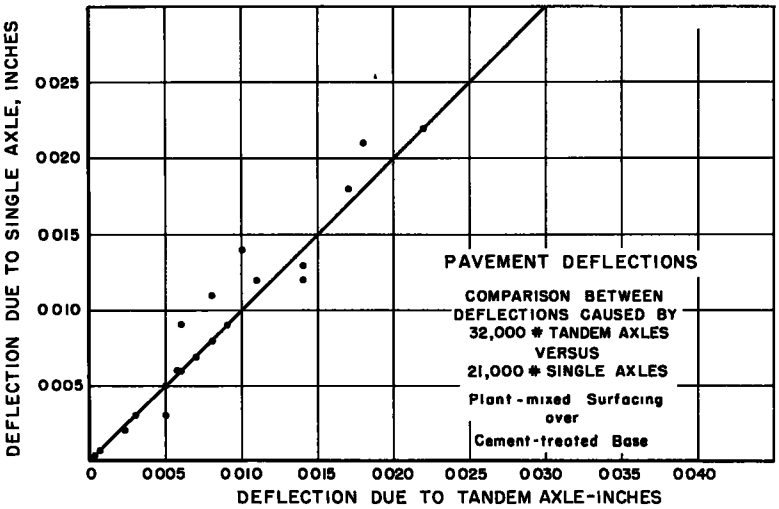


Figure 44.

This last relationship seems to be strikingly confirmed by an examination of the data for Road Test One-MD, where the destructive effect as indicated by the lineal feet of cracking produced by the 32,000-lb. tandem axles appears to be almost exactly equal to the amount that would be indicated by extrapolation for the same number of trips of a 24,000-lb. single-axle load.

Extrapolating the trend described above means that an analysis of strength requirements for a bridge deck should show still less difference between a load carried on single axles as compared to tandem axles. The relationship will vary with the length of the span, but according to the AASHO formula for a 32-foot span, a 32,000-lb. tandem-axle load should be equal to a 28,000-lb. single-axle load.

The interrelationship between load distribution and pavement strength is illustrated graphically by Figure 46. This chart indicates the "safe" deflections for several types of pavement and the relative deflection that would result for any condition from any change in load either single axle or tandem. The effect of slab strength is evident. Figure 46 is an attempt to rationalize the data where the deflections vary directly with load and indicates the orderly relationship between magnitude of load, axle spacing, pavement type, and deflections.

Mention was made earlier of the concept of stiffness and reference was made to work with the Shell vibration machine. In closing it should be mentioned that there appears to be some correlation between evaluations tentatively established by Nijboer and associates for pavements in Europe and the indications derived from deflection measurements in California. As outlined in the footnote, the concept of stiffness as used by Nijboer covers the total resistance of the pavement, base, and soil. This is also true of the deflection measurements as reported herein. Therefore, the relationship between stiffness and deflection may be expressed by the formula

$$S = \frac{L}{d}$$

Where S = the stiffness in kip per inch
d = the deflection in inches
L = the wheel load = $\frac{\text{axle load}}{2}$

Figure 47 shows the relationship between axle loads, deflections, and computed stiffness. As stated previously and indicated in Table 1, we have tentatively assigned limits for the deflection that pavements of various thickness and type will safely withstand over a period of years. These values, of course, are only tentative at this time, but it appears that the heavier pavements should be limited somewhere between 0.012 inch and 0.020 inch.

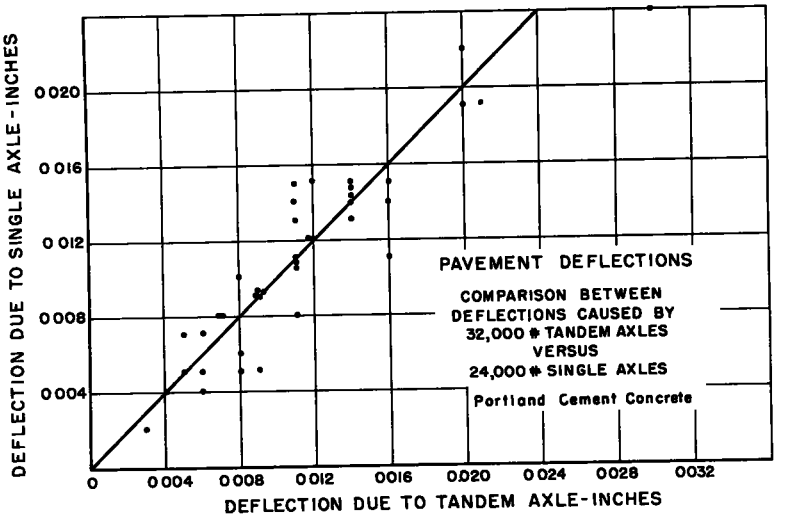


Figure 45.

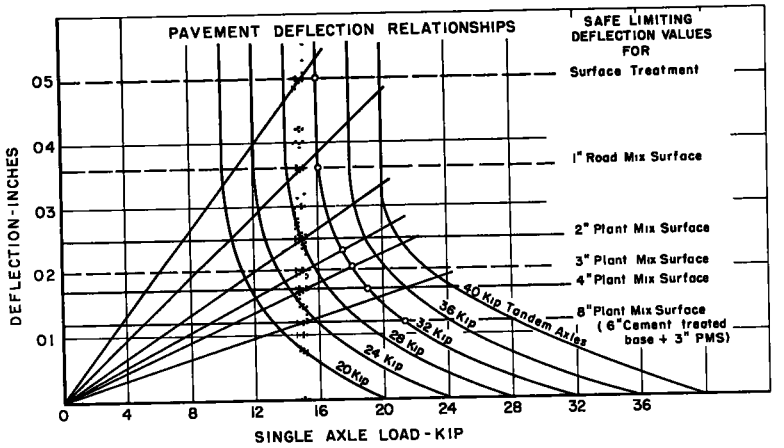


Figure 46.

Referring to Figure 47, this range of deflections is the equivalent of a stiffness factor in the approximate range of 400 to 600 kip per inch. For comparison, the limiting value of stiffness suggested by Nijboer and van der Poel (2) ranges from 570 to 1,140 kip per inch. This agreement is not too close, but the comparison is offered primarily to indicate that our present ideas of a limiting deflection are liberal rather than otherwise. If we accept Nijboer's conclusions, permissible deflections would range between 0.006 inch and 0.013 inch.

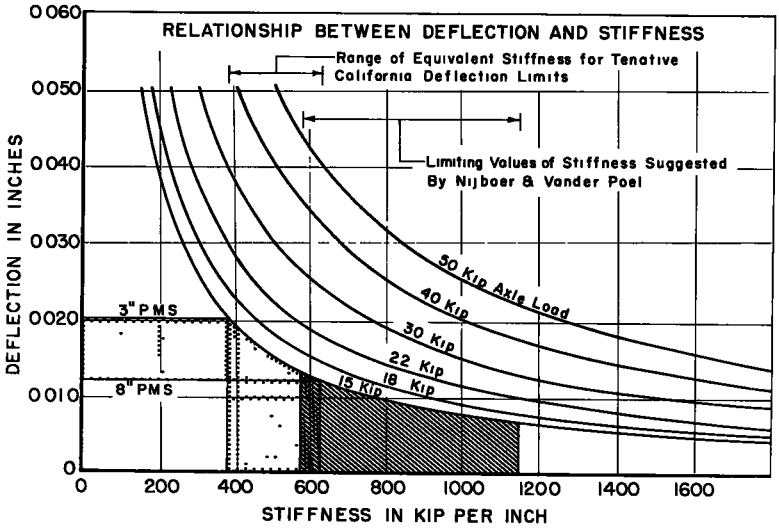


Figure 47.

CONCLUSIONS

In conclusion then it may be stated that there is an unusually close correlation between observations of cracking and fatigue type failures in bituminous pavements and the measured deflections which the pavement must undergo with each passing wheel load. These deflections appear to be associated with compression and rebound in the soil, and it is obvious that most pavements will withstand a few such deformations if not too often repeated. It may be said that a principal destructive force is the energy stored in the subgrade by each passing wheel.

It appears that the problem has three possible solutions: (1) provide a pavement[5] or wearing surface layer that is sufficiently flexible to accommodate repeated substantial vertical deflections without serious cracking; (2) decrease the magnitude of the vertical deflections to a tolerable limit by providing greater stiffness by means of greater depths of granular bases and subbases under the pavement; or (3) provide a pavement with a slab strength sufficient to sustain the forces induced by traffic which cause cracking.

With regard to Solution 1, above, the only method of utilizing flexibility, at present, is to use a thin bituminous surface treatment as there are no materials available today at reasonable cost to construct truly flexible surfaces of substantial thickness. Such thin "pavements" are, of course, vulnerable to other destructive effects of heavy traffic and to adverse weather conditions. Also, a thin surface is not able to provide the necessary strength or weight to carry loads over cohesionless sands or over plastic soils.

Solution 2 is in recognition of the fact that the magnitude of the deflections are related to the overall pavement structure thickness and that actual deflections can be reduced to an acceptable limit simply by increasing the thickness of a non-resilient base or subbase. At present, pavement and base thickness design procedures are predicated primarily on ability to carry loads over plastic soils (i. e., on resistance values and expansion pressures), which means providing sufficient cover thickness to support traffic over soils of low bearing or resistance value.

It appears that it will now be necessary to develop a second pavement structural design procedure based on resiliency factors and fatigue susceptibility in which soil resilience, magnitude of loads, load repetition and the stiffness of the pavement and base are all related in order to provide an adequate design. Both procedures will then have to be considered, and the thickest of the two pavement sections selected.

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[5]The term "pavement" as used herein, includes that portion of the overall pavement structure lying on the base and which is generally a mixture of aggregate and asphalt or portland cement.

It appears to be true, fortunately, that in many cases (perhaps in a majority of instances) low resistance values and high resilience characteristics go hand-in-hand; consequently, most pavements designed on strength factors are adequate from the standpoint of resiliency effects. However, there is evidence that this is not always the case, and there does exist an element of doubt which should not be allowed to exist if it can be eliminated.

Solution 3 calls for a pavement of considerable slab strength, such as heavy portland-cement-concrete slabs or the use of cement-treated bases in conjunction with a substantial thickness (4 inches minimum) of asphalt pavement. It has been observed, and substantiated to a large extent by the data included in this paper, that pavements or pavement structures of high slab strength need not be as great in overall thickness as sections utilizing lower slab strengths in order to perform satisfactorily in carrying traffic over resilient soils. However, for modern industrial traffic all sections must still be of substantial thickness.

It appears that the engineer is faced with the necessity of designing pavements of the various types described above in order to meet all three primary problems: (1) potential expansion, (2) plastic deformation, and (3) the resilience of the underlying materials. After designing comparable sections which will satisfy the above structural and physical requirements, it is then the engineer's responsibility to make an economic analysis to determine which one should be specified for a given location.

None of the foregoing promises to make life any simpler for those who must design highway or airport pavements.

ACKNOWLEDGMENT

The foregoing represents selected examples and extracts from data obtained in an extensive investigation which has been under way for over four years; so it is difficult to list by name all of those who have been involved and who have contributed time and effort to the various phases of this work.

Most of the studies have been carried on in the Pavement Section of this laboratory under the direction of Ernest Zube. The principal task of correlating and assembling the data, analyzing and studying the results has been under the direction of George Sherman, assisted by Robert Bridges. The cutting of cores, securing soil samples and installing gauge units were handled by Charles Clawson. James E. Barton and assistants were responsible for the General Electric travel gauge, electronic equipment, and recording of the deflections.

I should like to acknowledge the courtesy and assistance of the Shell Oil Company, which made available the Shell vibrator and the services of a skilled operator in order to compare "dynamic" deflections and other measurements without the necessity for constructing such an expensive unit.

The author has appreciated helpful discussions with W. N. Carey, Jr., of the Highway Research Board, and A. C. Benkelman, of the Bureau of Public Roads.

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Discussion

EARL C. SUTHERLAND, Bureau of Public Roads—The paper by Hveem deals principally with pavements of the flexible type but contains also some data and comments pertaining to rigid pavements. This discussion relates primarily to that part of the paper which discusses rigid pavements, but some data from tests on flexible pavements will be presented.

During the past 35 years a large amount of work both of a theoretical and an experimental character has been done in this country and in several European countries on the subject of the design of rigid pavements. A splendid summary of the researches on this subject, during this period, is contained in a publication (in English) by the Swedish Cement and Concrete Research Institute at the Royal Institute of Technology in Stockholm, 1949 (1).

The first comprehensive theoretical analysis of the stresses and deflections caused by loads acting on rigid pavements was made by Westergaard in the middle 1920's (2). In this investigation three cases of loading were investigated as follows: (1) corner, a wheel load acting close to a rectangular corner of a large panel of a slab; (2) interior, a wheel load acting at a considerable distance from the edges; and (3) edge, a wheel load acting at the edge but at a considerable distance from any corner.

In this study the influence of the size of the loaded area was investigated and full subgrade support was assumed for all cases of loading.

During the 1930's the Bureau of Public Roads carried out a series of experimental researches, known as the Arlington investigation, one phase of which was devoted to a study of the various aspects of the Westergaard analysis. The results of this investigation were published in 1943 (3) and showed that the stresses and deflections computed by the Westergaard formulas were in close agreement with the stresses and deflections determined by measurement for the interior and edge cases of loading, but were somewhat smaller than the critical measured values for the corner case of loading. The lack of agreement for the corner loading was due to the fact that the ends and corners of a pavement slab warp upward during the night and thus do not have the perfect subgrade support assumed by Westergaard.

During the Arlington tests a large number of tests with the corner loading were made at night when critical upward-warping conditions prevailed. The data obtained were used to develop an empirical modification in the Westergaard corner formula. The corrected formula is sometimes referred to today as the BPR or the Kelley corner formula (3, 4). These same data were used also in the development of both the Spangler and Pickett empirical corner formulas, the latter being that recommended by the Portland Cement Association (5).

The theoretical and experimental studies of the design of rigid pavements made up to this time are of significance with respect to the paper under discussion.

In Table 1 the author gives what are termed "Maximum Permissible Deflections for Design Purposes (Tentative)". The deflection value for concrete pavements, 0.012 inch, was determined for the interior case of loading and was apparently selected because the subgrade acts in a manner highly elastic, or resilient, as it is termed in this paper, at the interior.

Cracking in concrete pavements is, of course, related to the stresses and the data obtained in the earlier work referred to above shows that the deflections of a concrete pavement slab caused by loads at the interior and edges are of little significance as an indication of the stresses produced by the loads (2, 3). For example, for these two cases of loading the size of the bearing area has a negligible influence on the deflections but has an important influence on the stresses caused by loads. However, for the corner loading the deflections, under some conditions, are of significance in predicting stresses.

While deflection values are of little significance as a measure of the magnitude of the stresses that might be caused by loads acting on a concrete pavement, their magnitude is important with respect to the development of pumping and consolidation of the subgrade. The most-serious structural damage that results from pumping is a slab-end failure where transverse cracks develop at 6 to 8 feet from the slab end. This failure is not

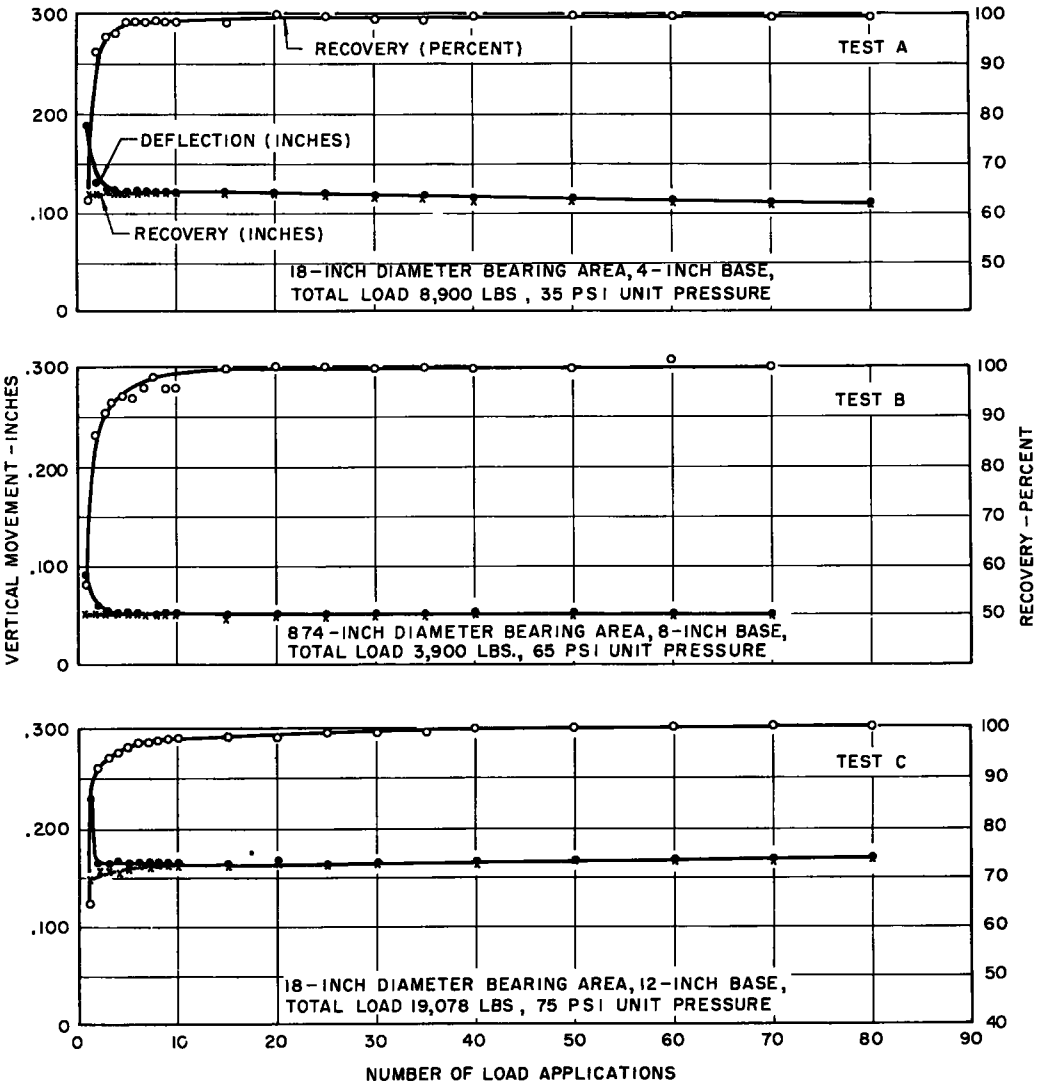

Figure A. Repetitional load tests on flexible pavement bases of different thicknesses, individual deflections and recoveries.

caused by any of the cases of loading analyzed by Westergaard, although in some respects it resembles the corner case of loading.

In the Arlington investigation (3) tests were made to study the elastic action of the subgrade under concrete pavements and also the magnitude of the deflections that develop under normal loading for the different cases analyzed by Westergaard. It was found that for the edge and interior cases of loading the subgrade acted elastically while at the corner (6) it did not. This is in agreement with the author's findings. It is probable that a reduction in the magnitude of the corner deflections would result in more nearly elastic action of the subgrade in the vicinity of the corner. However, if the pavement were to be designed with sufficient thickness so that the deflection at the corner will not exceed approximately 0.012 inch the result would be an unduly thick pavement.

The deflection data obtained in the Arlington tests were for static loads of a magnitude that would develop transverse bending stresses of half the modulus of rupture of the concrete. With this criterion for load magnitude it was found that the ranges in deflections for the different cases of loading on slabs 6, 7, 8, and 9 inches in thickness were

about as follows: corner loading, 0.040 to 0.055 inch; interior loading, 0.008 to 0.012 inch; and edge loading, 0.018 to 0.020 inch.

Based on certain deflection data obtained in tests on concrete pavements (see Figure 45), the author makes the statement that a 24,000-lb. single-axle load is equivalent to a 32,000-lb. tandem-axle load. While not definitely so stated, these deflection data appear to have been obtained from tests at the interior of the pavements. It is stated that this selection of equivalency is supported by the crack data obtained in the Road Test One-MD investigation (7).

Except for a moderate amount of longitudinal cracking which occurred in Section 4, all of the cracking that developed in Road Test One-MD was in the vicinity of the slab ends and was caused by a slab-end loading, (called a corner loading in the report). This cracking appeared only after appreciable pumping had developed in the vicinity of the slab ends.

As pointed out earlier, the deflections caused by loads acting at the interior of a concrete pavement slab bear no direct relation to the magnitude of the stresses caused by the same loads. Furthermore, since cracking in the Maryland road test developed only after bad pumping had occurred, it is the writer's opinion that the amount of cracking should not be used as a criterion of load equivalency, particularly when applied to modern pavements where effective provisions have been made to control pumping.

In the report on Road Test One-MD, load-stress and load-deflection curves are pre-

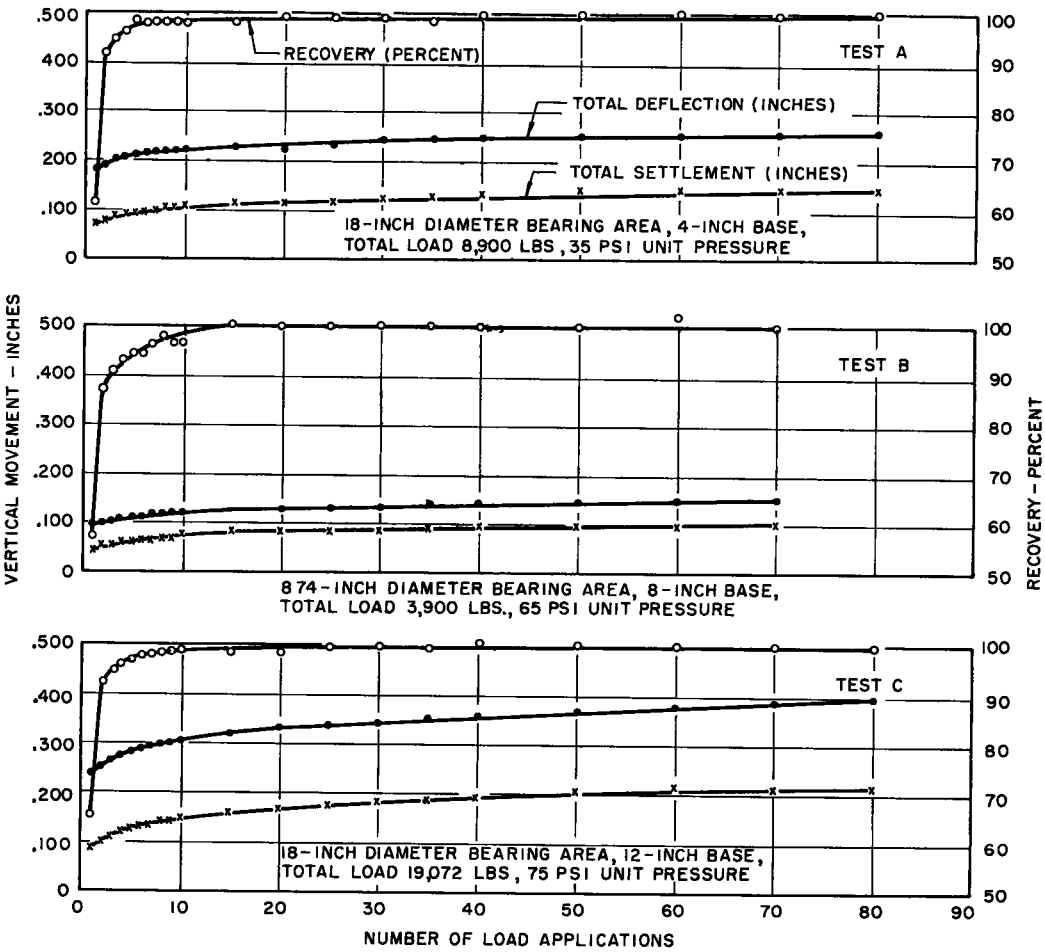

Figure B. Repetitional load tests on flexible pavement bases of different thicknesses, total deflections and total settlements.

sented for all of the cases of loading investigated for both the granular nonpumping soil and the pumped fine-grained soil. These curves are plotted in a manner such that equivalent single- and tandem-axle loads can be determined.

The interior loading is not critical and should not, therefore, be used for the selection of equivalent single- and tandem-axle loadings. As stated earlier, slab-end failures, caused by what was termed corner loading in the Road Test One-MD report, appear to be the most-serious type of structural damage to be found in concrete pavements today. For this type of loading on the granular nonpumping soil it is found, Figure 87 of the report, that at creep speed the 32,000-lb. tandem axle is equivalent to an 18,000-lb. single axle (7).

The author presents certain data pertaining to the "resilience" of flexible pavements and discusses this subject at some length. The Bureau of Public Roads has investigated this subject in a number of tests made over a period of years, and samples of the data obtained may be of interest in this discussion. However, the term "elastic action" rather than resilience has been used by the bureau. The object of the tests was to determine whether, under applied loads, the elastic action of the various components of a flexible pavement could be used as a criterion for determining the load carrying capacity of the structure. Flexible pavements carrying a large amount of traffic must act nearly elastically if rutting is to be avoided and the pavement is to remain smooth.

Figures A and B show samples of data obtained in tests made in 1946. The subgrade under the pavement on which these tests were made was of the A-6 group and on it there were three thicknesses of gravel base, 4, 8, and 12 inches, respectively.

As indicated in the figures, 70 to 80 static loads were applied on top of the base in each test. The loads were maintained for 2 minutes, and 2 minutes were allowed to elapse after the removal of each load before applying the next. As indicated on the graphs, different diameter bearing areas and different unit pressures were used.

In Figure A the deflections and recoveries are plotted with respect to a base measurement made immediately before the application of the respective loads, while in Figure B they are plotted with respect to a base measurement made at the beginning of the load test series. Thus, Figure A shows the deflection and recovery for each load of a series, while Figure B shows the total deflections and settlements caused by the series of loads. The same data were used in constructing both graphs.

The tests included in these two figures were selected as being representative of those in which the pavement appeared to act nearly elastically. It will be noted that 100-percent recovery was indicated for all three tests after a considerable number of loads had been applied.

It may be observed in Figure A that there is a marked reduction in the deflections and an increase in the recoveries for the first 10 to 15 load applications, after which they remain approximately constant and equal.

The manner of plotting in Figure B is more sensitive than that used in Figure A, as it shows a progressive increase in the total deflections and total settlement, even though essentially complete elasticity is indicated on the basis of individual load applications. From Figure B it must be concluded that completely elastic action was not attained in any of the tests.

Figure C shows data from tests in which the loads caused definite failure of the pavement. The data of this figure were plotted in the same manner as in Figure A.

The data of Figures A and B show that, to determine when a pavement is acting in an essentially elastic manner, it is necessary to measure the deflections and settlements for a large number of repeated loads. If the deflections are to be measured at the level of only one component of the pavement structure it would seem most logical to measure the movement at the top of the base as was done in these tests.

If a pavement is to continue to act elastically under repeated loadings, it is necessary that the magnitude of the deflections be sufficiently small that the structure of the various supporting components will not be broken down. The author presents certain data on this point with respect to the pavement surface. It would appear to be desirable to so design the pavement surface as to enable it to withstand a greater degree of flexing, if this could be accomplished without sacrificing stability. This would make it possible to utilize the supporting power of the subgrade to a greater degree and might lead to

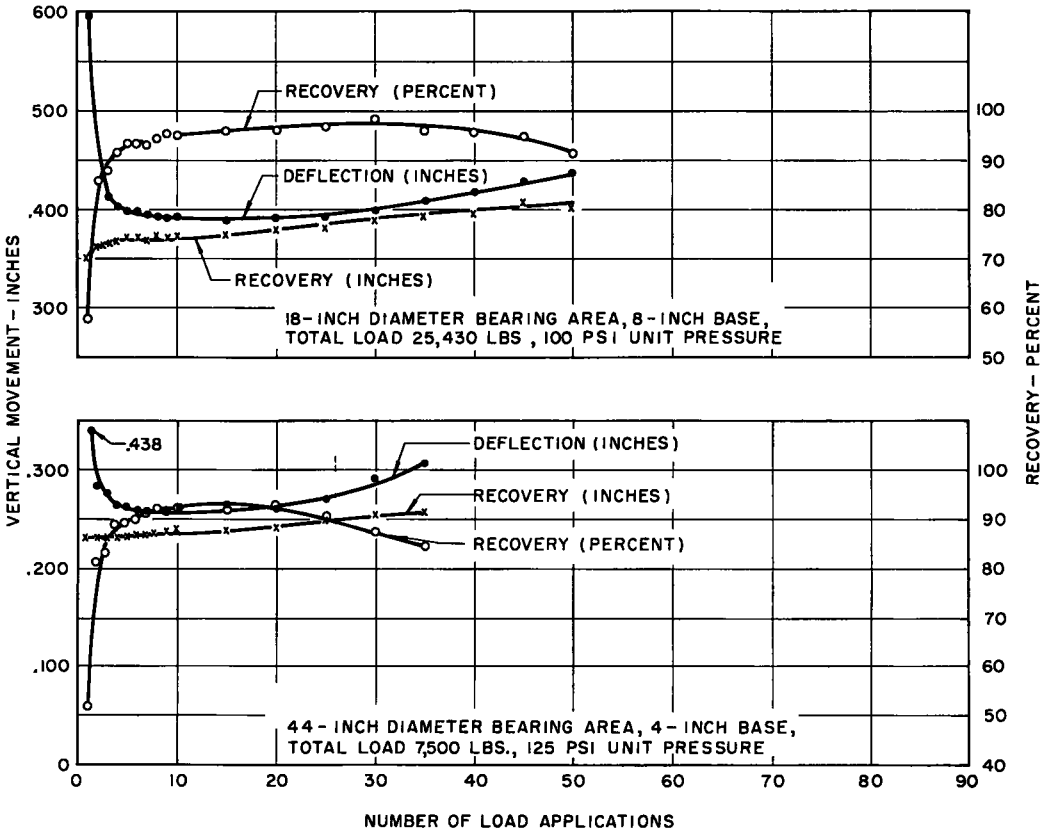

Figure C. Repetitional load tests on flexible pavement bases of different thicknesses, individual deflections and recoveries.

more-economical designs.

Investigations of flexible pavements in the past have been greatly handicapped by a lack of a dependable criterion for evaluating their load-carrying capacity. Elastic action might prove to be a good criterion for high-type flexible pavements, such as are used on primary highways. This subject should be pursued further for both repeated static and repeated dynamic loading.

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W H. CAMPEN, Manager, Omaha Testing Laboratories, Omaha, Nebraska—Hveem has presented some interesting and provocative data. I wish to make the following comments.

Deflection or elastic deformation has been recognized as a factor in the designing of pavement thickness. For instance, the designers of concrete pavements select thickness on the basis of the subgrade modulus of reaction which involves the consideration of deflection. Furthermore, some designers of flexible pavements select thickness on the basis of deformation obtained by the use of plates. This deformation includes deflection.

Hveem has found that flexible pavements topped with a 2-inch bituminous surface are about twice as flexible as concrete pavements. This relationship seems to be consistent with expectations. However, the magnitude of the deflections which cause failure seem to be much lower than expected. For this reason it would be advisable to make deflection measurements over a wide area and on a wide variety of pavements before we draw definite conclusions.

Incidentally, the field deflection measurements could be made easily with the Benkelman beam. The use of this device would eliminate the possibility of errors due to the location of the reference point in the Hveem method. Hveem's data show that at any one location the deflection increases as the reference point is moved downward to depths of 18 feet and more.

The increase in deflection with the lowering of the reference point indicates that the effect of loaded surface areas extends to depths equivalent to several times the diameter of the loaded areas. This is significant to those who used plates for the evaluation of soils for foundation purposes. One of the principal objections to the use of plates for this purpose has been that the effect does not go far enough to engage the deeper soils.

Hveem reports that, in the testing of a plastic soil by the resiliometer, compressibility increased as water content was increased above the optimum. This result is at variance with basic fundamental principles. I hope he will determine whether the compressibility is due to entrained gases or the soil particles themselves. No doubt it can be assumed that water is incompressible.

I believe it can be shown that the deflection of a layered system is due principally to the soils beneath the base and that the deflection is usually due to the compressibility of entrained gases. Furthermore, the amount of the entrained gases is variable. Based on these assumptions, how can Hveem's resiliometer results on a small sample of soil in the laboratory be used to calculate deflections in the field?

STUART WILLIAMS, Highway Physical Research Engineer, Bureau of Public Roads and A. W. MANER, Assistant Testing Engineer, Virginia Department of Highways—Hveem is to be commended for his progressive thinking and action in connection with the problems of structural design of pavements. The work conducted in California under his direction serves to emphasize the great importance of the elasticity or resilience of the subgrade soil in the problem. It also points to the fact that information of considerable value can be obtained from deflection studies of flexible pavements in service.

In this connection, the results of a load-deflection study conducted cooperatively by the Virginia Department of Highways and the Bureau of Public Roads on a flexible pavement of modern design are of interest. The pavement, 5 miles in length, is located on State Route 7, east of Leesburg, Virginia. The old road on this location had a poor performance record which was attributed largely to the type of subgrade soil existent over much of its length. Accordingly, it was decided to rebuild rather than to strengthen the existing pavement.

The new pavement was designed by means of the Virginia method which is based on the CBR test with certain modifications[1]. Location is generally along the old route, but numerous changes in both horizontal and vertical alinement were made. Construction was completed in 1951.

A typical transverse section is shown in Figure A. Modified CBR values of a number of subgrade soil samples, tested in the laboratory, ranged from 1.5 to 17.0. Borrow material having a CBR of about 8 was readily available near the job and was used for

[1] "Designing Flexible Pavements (Virginia)" by D. D. Woodson. Research Report 16-B, Highway Research Board, 1954.

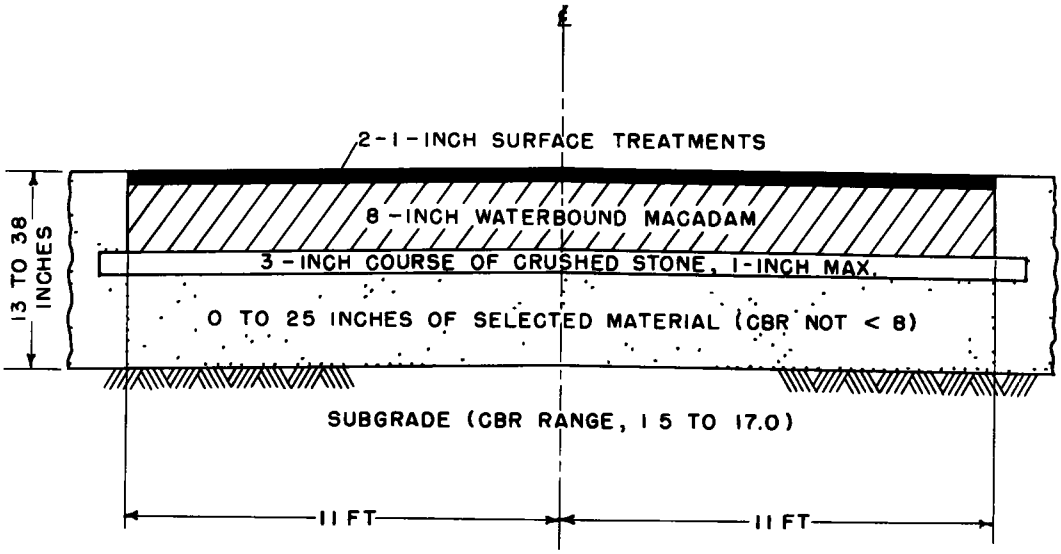

Figure A. Typical transverse section showing components of the overall pavement structure.

subbase in thicknesses of 0, 6, 12, 22 or 25 inches. The base course, uniform throughout the length of the project, consists of an 8-inch thickness of waterbound macadam constructed in two 4-inch courses. Underlying the base is a 3-inch course of crushed

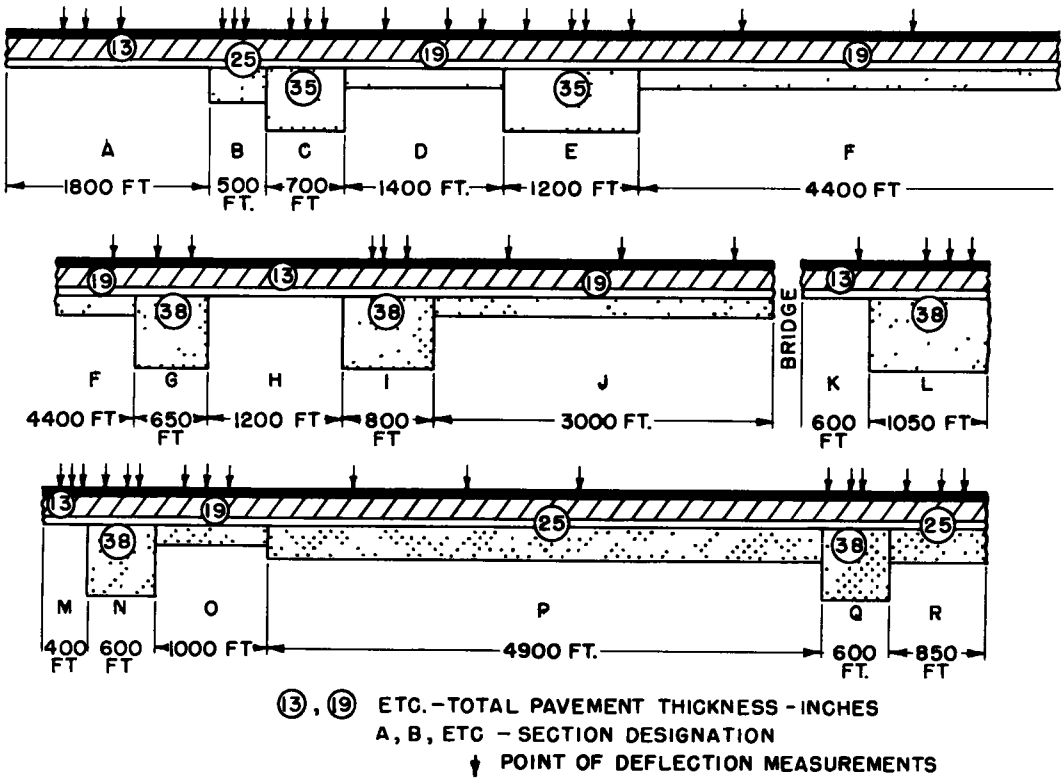

Figure B. Longitudinal section showing locations where deflection measurements were made.

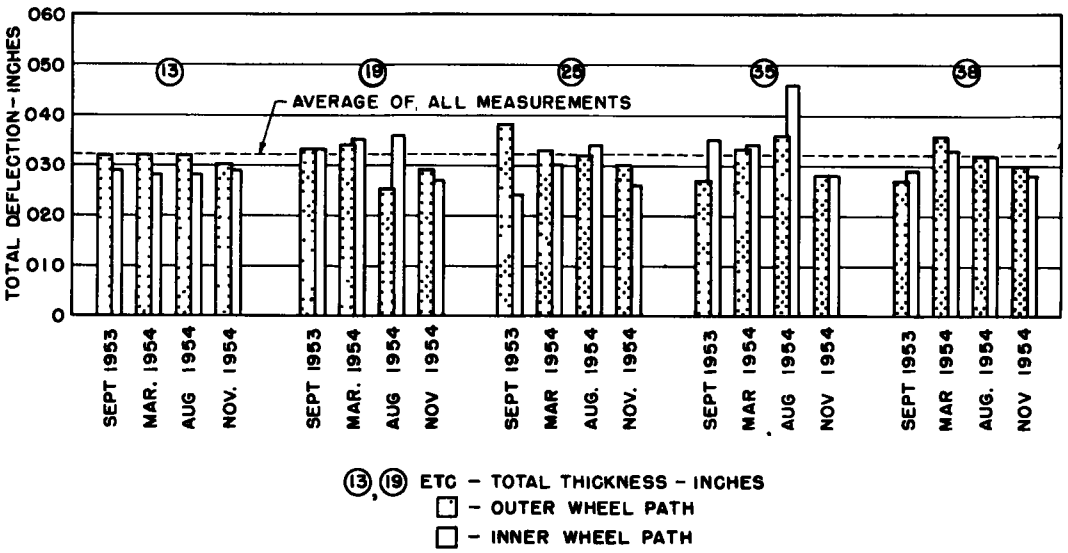

Figure C. Deflections obtained at different times of the year on the various pavement thicknesses.

stone, 1-inch maximum size. A surface treatment or armor coat about an inch thick was laid soon after completion of the base course. This was reinforced by a second similar treatment about 2 years later. Subgrade CBR test values, the corresponding five pavement design thicknesses constructed and the total length of pavement of each thickness are listed in Table A.

TABLE A

CBR test value	Total pavement thickness	Total length of pavement
	Inches	Feet
1.5	38	3,700
1.8	35	3,700
3.7	25	6,300
5.5 - 6.7	19	9,800
10.0 - 17.0	13	1,800

Additional information regarding the length, thickness and location of the individual pavement sections is shown graphically in Figure B. The sections, lettered A to R, range in length from 400 to 4,900 feet. Also shown in Figure B are the points at which deflection measurements were made. In general, measurements were obtained at three arbitrarily selected points in each section.

This investigation was conducted to: (1) obtain an indication of the deflection of the pavement as a whole; (2) develop information concerning the relative strength of the five different overall pavement thicknesses; (3) determine the uniformity in strength within each section of pavement having the same overall thickness; (4) compare the indicated strength of the pavement in the outer wheel path (edge of pavement) with that of the inner wheel path (interior of pavement); (5) study the elastic action of the pavement, i.e., to determine whether the application of the test load produces any permanent movement; and (6) determine the effect of seasonal changes on deflection.

The load-deflection tests were made in the fall of 1953 and in the spring, summer, and late fall of 1954. One day was required to complete the tests for each of these periods. A heavy duty two-axle truck equipped with 11.00-by-20 dual tires was used to apply a 9,000-lb. wheel load to the pavement at creep speed (1 to 3 mph.). The truck traveled in the eastbound lane with the centers of the rear wheels approximately 1.5 and 7.5 feet from the edge of the pavement. The lines of travel of the outer and inner wheels are referred to as the outer and inner wheel paths.

Pavement deflections were measured with the Benkelman beam deflection indicator which was developed for the WASHO Road Test in Idaho[2]. By means of this device a comparatively large number of load-deflection and recovery measurements were obtained in a relatively short period of time. At each location a single measurement was

[2] "The WASHO Road Test", Special Report 18. Highway Research Board, 1954.

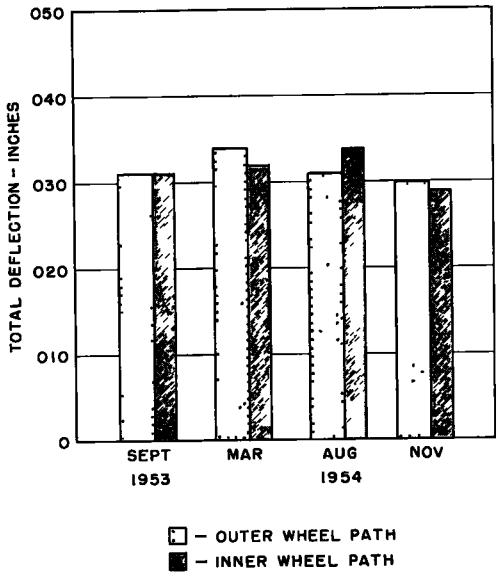

Figure D. Comparison of pavement deflections obtained at different times of the year. Average of all measurements for each test series.

made between the rear dual tires in each wheel path as the vehicle moved forward at creep speed.

From the structural viewpoint the pavement has performed well since it was rebuilt. During the first 2 years spot patching of a number of small areas of distress in the temporary surface treatment was required. It is believed, however, that this distress was due to raveling of the temporary surface treatment, rather than to structural instability of the pavement. At two of the deflection-test locations, both of which are on the poorer subgrade, some general cracking of the temporary surface, particularly near the pavement edge, had occurred. In order to augment the wearing course and to improve the smoothness of the pavement, a second surface treatment was applied over the entire project in the fall of 1953.

Traffic on this pavement in 1954 consisted of about 3,200 passenger cars, 125 tractor-trailer combinations and 525 light to medium-weight trucks per day.

Although this study should not be considered a comprehensive one and, as has been mentioned previously, consumed a minimum of time, several interesting findings have resulted. For any one test period the individual deflection values vary considerably between sections of the same overall thickness (about 0.025 inch) and even between points in the same section (about 0.015 inch).

The permanent vertical movement caused by the application of the one load of each of these tests ranges generally between plus and minus 0.002 inch. In over a third of the total tests made, however, the deflection and recovery are equal.

The averages of all measurements in the two-wheel paths for each of the overall pavement thicknesses are shown graphically in Figure C for the four seasonal test series. The individual bars represent averages of from 3 to 14 readings, the majority being an average of 6 or more. The following general comments may be made concerning these average values:

1. The deflection of the entire pavement for all test periods is 0.032 inch.
2. With the exception of a few erratic values, the deflection does not vary appreciably with change in pavement thickness. Values range from about 0.025 to 0.035 inch.
3. Comparatively little effect of seasonal changes is indicated, except that for all pavement thicknesses late fall deflection values (November) are somewhat smaller than those of the remaining test periods.
4. The difference between the deflections of the outer and inner wheel paths shows no consistent trend except for the 13-inch-thick pavement, where those of the outer wheel path are somewhat the greater.

In Figure D the average deflections for each period of testing are shown, each bar representing the average of all measurements in each wheel path. On the basis of these values, the largest deflections were obtained in March, although they are only slightly greater than those of the other periods. The minimum deflections occurred in November. It is apparent that seasonal changes had only a small effect on pavement deflections. It should be pointed out, however, that the winter of 1953-54 was comparatively mild and the precipitation during the period of this study was below normal. The deflections in the outer wheel path for the March and November tests are somewhat greater than those of the inner wheel path. However, this relation is reversed for the August tests, while in September the deflections in the two wheel paths are equal.

SUMMARY

The average of all the deflection measurements made on the Leesburg pavement is 0.032 inch, with comparatively few individual measurements greater than 0.040 inch. Performance under moderate traffic has been excellent; therefore, the pavement as constructed might be considered adequate. According to the data presented by Hveem, the safe deflection limits for an 18,000-lb. single-axle load would range from 0.030 inch for a 2-inch plant-mix surface to 0.043 inch for a 1-inch road-mix surface.

Based on both visual inspection and average deflection measurements, there is little difference in the behavior of the sections of different overall thickness. In view of the fact that the subgrade consists of soils having a great range in load-bearing values, this indicates that a rather uniform strength pavement was obtained by the design method used.

It was found that the individual deflections as measured at the different locations in sections of uniform thickness varied to a considerable extent. Consequently, for a study of this nature a great number of measurements should be made so that local inconsistencies will have a minimum effect on the results. With the deflection measuring device used in this investigation, this is possible.

The pavement appears to react to load in a nearly elastic manner. At some locations the "permanent" displacement caused by the test load was slightly downward and in others slightly upward, but in over a third of the tests the recovery was equal to the deflection.

Seasonal changes caused little effect on pavement deflections. It might be expected that deflection values would decrease from a maximum in March to a minimum in November. Minimum deflections were measured in November, although they were only slightly less than those in March. The maximum movements occurred in the summer. Possibly the somewhat abnormal climatic conditions existing for the period of the study contributed to this result.

The assistance of A. W. Furgiuele, Materials Engineer, Culpepper District, Virginia Department of Highways, in the conduct of the deflection tests is gratefully appreciated.

F. N. HVEEM, Closure—Sutherland gives an outline of some of the investigations and theoretical analyses relating to stresses and deflections of rigid pavements. He points out that the Arlington investigation conducted by the Bureau of Public Roads failed to confirm the Westergaard formulas for computed stresses and deflections at the corners and ends of slabs.

An extensive field investigation of concrete pavements in California conducted some ten years ago partially reported in ACI "Proceedings" for 1951 Vol. 47, p. 797 supports Sutherland's statement about warping slabs. But in passing, I might offer the opinion that undue emphasis has been placed on the fact that the corners show the greatest amount of warping. In the hours between 5 A. M. and 7 A. M. the greatest curling is generally developed, the entire ends of each slab being lifted from the subgrade for a distance ranging from five to seven feet from the end of the slab, (see Figure 19 of ACI paper and see Figures A and B of this closure).

Repeated flexing of this "cantilever portion" of the slab accounts for the transverse crack which Sutherland mentions. These cracks develop along an irregular line about 6 to 8 feet from the slab end. Sutherland is correct in stating that this type of failure is not explained by the Westergaard analysis, but it is also true that this type of transverse failure rarely develops unless the subgrade soil is washed out or eroded away by the pumping action of the "curled up" free end of the slab. While slabs curl and "pump" on our cement treated subgrades they do not show the transverse crack and faulting is negligible.

In the measurement of deflections on California pavements we were not primarily interested in measuring the vertical movement at the slab ends as the amount of this movement varies greatly throughout the day, as shown by Figures A and B, and is not consistently or primarily due to resiliency of the underlying soil or foundation.

Sutherland states "It is probable that a reduction in the magnitude of the corner deflections would result in more nearly elastic action of the subgrade in the vicinity of the

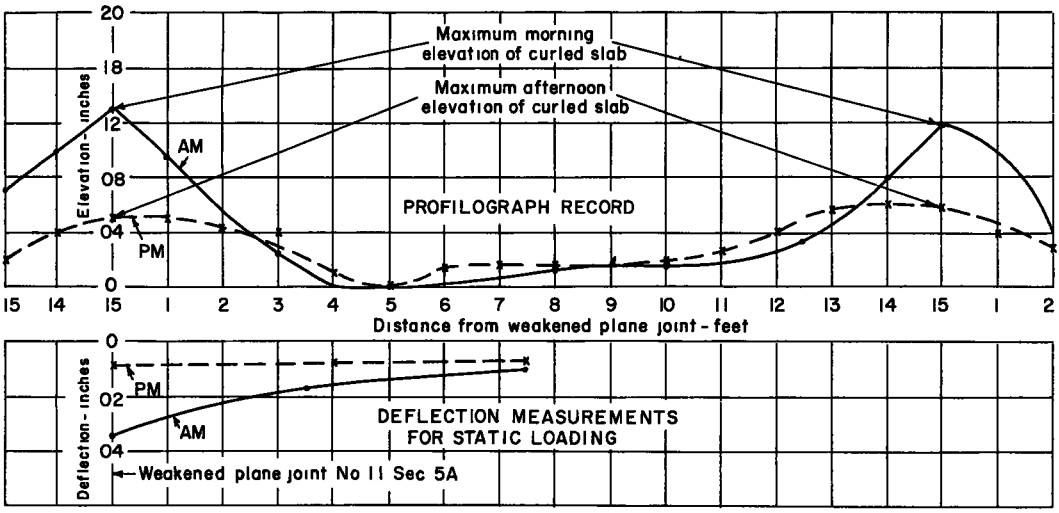

Figure A. Comparison of profilograph record and deflection of slab due to static load.

corner." Figures A and B both indicate that when the curling of the slab is diminished or flattened by the expansion of the upper surface due to direct heat from the sun the slab is then more nearly in contact with the subgrade throughout its entire length and when this condition exists the deflections measured at the end of the slab were found to be little if any greater than at the center.

If it were generally true that the unsupported ends of concrete pavement slabs transmitted significantly greater loads to the subgrade, then the subgrade soil near the ends of the slab should be compressed or compacted to greater density and this should be measurable if any significant compression resulted. Our study of concrete pavements indicated that the reverse is generally the case. In fact, all soils (with the exception of dry cohesionless sands that consolidate from vibration) showed less average density under the slab ends when compared with the average density of the soil under the center of the slab.

A glance at Figures A and B (where it is shown that the slab is in continuous contact with the subgrade only at the mid-point) indicates why this condition should exist. Therefore, in our study of deflections which were primarily aimed at determining the effect of

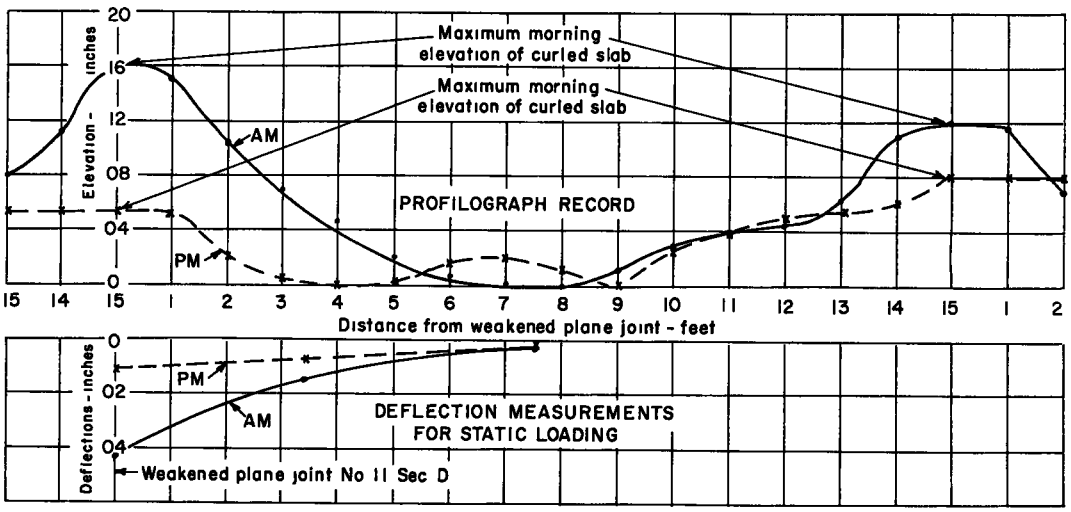

Figure B.

resilience in the soils, we felt that it was necessary to confine the deflection measurements to the midpoint of the concrete slabs.

As illustrated in Table 1, we reached the tentative conclusion that a safe limit for concrete pavement when resting on the subgrade should not greatly exceed 0.012 inch for heavy traffic, and Sutherland indicates that for interior loading the deflections for heavy slabs under loads that would develop transverse bending stresses of half the modulus of rupture were in the order of 0.008 to 0.012 inch. From our viewpoint, this is excellent agreement and it would seem that the data cited by Sutherland tends to support the tentative conclusions suggested by our work. In other words, for an admittedly limited comparison the agreement appears to be surprisingly good.

Sutherland takes the stand that the cracking developed on Road Test One-MD is not a proper measure of load equivalency, because cracking developed only as an aftermath of pumping. He states, however, that the pumping was the result of excessive deflection. He would probably also agree that cracks did not occur with the passage of a single truck but developed with the repetition of loads.

It should be pointed out that pumping action, if we mean the intake and forceful ejection of water from beneath the slabs, could not of itself cause cracking. The pumping of concrete-pavement slabs is significant only to the extent that it removes the subgrade support by washing away the soil, leaving a cavity that permits greater deflection. Therefore, it seems difficult to avoid the conclusion that excessive deflection coupled with load repetition was the primary cause for the cracking on the Maryland road.

Reports on the concrete pavement test by the Corps of Engineers at Columbia, Mississippi, seem to show a similar pattern in which the linear feet of cracks produced in the pavement by the test vehicle shows a relationship both to the magnitude of the load and to the number of load repetitions.

It is our understanding that the Maryland road was selected for test purposes because it was considered to be representative of a large mileage of pavements in the United States. Figure 45 in the subject paper shows that the deflections of a rigid pavement under 32,000-lb. tandem-axle loads were approximately the same as would be produced by a 24,000-lb. single-axle load, and the published data from the Maryland test road indicated that the linear feet of cracking developed there was the same for the 32,000-lb. tandem as for a 24,000-lb. single axle. It was my intention merely to point out this similarity, but I was not ready to propose a method of design that would make the tandem axles less productive of deflection and pavement cracking.

Sutherland points out that deflections caused by loads acting at the interior of a concrete pavement slab bear no direct relation to the magnitude of the stresses caused by the same loads. It is Sutherland's opinion that the amount of cracking should not be used as a criterion of load equivalency and prefers to focus attention upon the stresses rather than upon the amount of movement. No one will question the obvious fact that the breaks or cracks are associated with either the magnitude or repetition of stresses in the slab. Nevertheless, the application of orthodox structural design concepts to pavements has not seemed to be fruitful.

The results of the Arlington experiments were published in 1935-6. Since that time, a great many pavements have been designed and constructed in an effort to avoid the weaknesses shown by that study. But many of these "improved" pavements are deflecting excessively, either pumping or faulting; cracks are developing prematurely. It appears that many engineers are still dissatisfied with the results of much so-called modern pavement design. If this were not true, there would seem to be little justification for the $11-million test road currently proposed by AASHO.

On the subject of flexible pavements, Sutherland states: "The author presents certain data pertaining to the "resilience" of flexible pavements." This statement by Sutherland is not quite correct. In the first chapter of the paper, "resilience" was listed as one of the properties of the basement soil, and at no point did I intend to convey the impression that asphaltic pavements are considered to be resilient. Sutherland states that the Bureau of Public Roads has used the term "elastic action" rather than resilience. No one can question the validity of the term elastic in this connection; but as explained in a footnote in the paper, it is felt that the term "elastic," as commonly applied to structural materials, may be therefore appropriate for virtually all materials such as steel, concrete,

glass, etc. However, even engineers are aware that a sponge-rubber mattress feels different to lie on than a concrete slab. Both are elastic, but the concrete is not resilient. In this comparison, asphalt is undoubtedly less elastic than portland-cement concrete. The so-called flexible pavement with a high asphalt content is probably among the least elastic of structural materials.

In any event, there appears to be no essential disagreement between our findings and the data presented by Sutherland showing the compression and rebound characteristics of a base subjected to repeated loadings under bearing plates. These data are interesting; but in order to evaluate the conditions responsible for either good or poor pavement performance, it seems necessary that we reproduce the actual field conditions, the type of contact between the load and pavement, and by use of full-scale loads on pneumatic tires, determine the amount of deflection to which the pavement is subjected by the vehicles which it must sustain many times daily. Also, when load-deflection tests are performed on an existing pavement that has been under traffic for a period of time, it can be reasonably assumed that the pavement and base have already been "conditioned" so that the measurements will represent its current or "normal" behavior.

Sutherland states: "It would appear to be desirable to so design the pavement surface as to enable it to withstand a greater degree of flexing if this could be accomplished without sacrificing stability." We stated that this is one solution. However, there are two other possibilities. Another is to make the pavement and base of high-strength material which will reduce the deflections below a safe limit.

In fact, there are three possible solutions to this problem, as shown in the lower-right corner of Figure 2. One may design the pavement to develop a sufficiently high slab or flexural strength; the thickness of granular materials may be increased so that the weight tends to reduce any flexing or bending due to resilience in the underlying soil, and finally, a very-thin or flexible pavement may give excellent performance over such foundations.

Both theory and experience testify to the possibility of utilizing the solution mentioned by Sutherland. In fact, if the engineer had at his disposal a paving material that was truly flexible and sufficiently tough and durable, some radical changes in pavement design concepts would be possible. We are handicapped by the fact that we have no paving materials that are truly flexible, and the only means of utilizing this quality is to employ very-thin layers comparatively rich in asphalt such as surface treatments, armor coats, etc. Such thin surfaces are easily damaged by heavy traffic and severe weather conditions, but many have given a remarkable performance, testifying that the principle of a thin wearing surface is sound if it can be made sufficiently tough and durable.

I wish to thank Sutherland for his comments and for the interesting data which adds additional support to the observations discussed in the paper.

Campen notes that deflection or elastic deformation has been recognized as a factor in the designing of pavement thickness. He is correct in pointing out the similarity with the concept as expressed by Westergaard in terms of modulus of subgrade reaction. It is, however, not so evident that there has been orderly procedure for utilizing test data or for meeting this problem on any other basis than personal opinion or "judgment".

Campen has expressed surprise that a critical deflection is of such a low order of magnitude. This would seem to indicate that any "allowance" which has been made must have been based upon assumptions, and it is not clear that many design methods have made "allowance" for the effect of load repetition. Also Figure 14 of the paper illustrates that there may be a great difference between static and dynamic deflections.

Campen also comments that these data are "significant to those who used plates for the evaluation of soils for foundation purposes." In order to be applicable data secured by bearing plates or any other method should be based upon soils in the condition in which they will exist for a substantial period after construction. This means that one must virtually construct a portion of the project before knowing what will be entailed in the design; hence, it would be difficult to know whether the project could be financed until it has been constructed, and in the second place, it is also a most difficult matter to prepare even a small area of subgrade and produce in a short period of time the condition of moisture and density which may be typical of the soil after the passage of time.

Campen also states that the resiliometer results which indicate that compressibility

increases as the water content is increased above the optimum "is at variance with basic fundamental principles." As stated in the paper we were admittedly somewhat surprised to note this trend. Nevertheless, we have more faith in "basic fundamental principles," rather feeling that it is a contradiction in terms to imply that observed behavior is in conflict with fundamental principles. It seems safer to assume that we need to know more about fundamental principles, especially which principles are applicable, to explain an observed behavior.

Referring to Campen's final comment, we readily agree with the assumption that the compressibility of entrained gases is the most probable source of the resiliency noted. It also seems most probable that the nature or shape of the soil particles has some affect on the resiliency.

Even though this is true it does not appear unreasonable to hope that we could secure an indication or index by testing a relatively small sample of soil in the laboratory. Engineers have long been satisfied to base the design of steel or concrete structures on relatively small samples tested in the laboratory, even though there is ample evidence to show that as the size of the test specimen is diminished the unit tensile or compressive strengths indicated are usually much higher than can be expected in large members, such as beams or columns of the same material. This fact does not prevent engineers from making good use of test data on small specimens.

Maner has presented an interesting account of deflection measurements on the Virginia State Highway Route 7, east of Leesburg. It appears that there is surprisingly close correlation between the magnitude of deflections associated with a satisfactory pavement in Virginia compared with indications in California. Obviously a pavement might show substantial deflection under a 9,000-lb. wheel load when subjected to test, but the pavement as a whole should not be damaged if 9,000-lb. or similar heavy wheel loads were infrequent in the traffic pattern. As indicated by Maner, the high degree of uniformity in the deflections measured seems to indicate that they have executed an excellent design, as the variation and thickness of subbase evidently compensates for variations in the character of the basement soil.

It would be interesting to examine these deflection data further by reference to the length of the depression created by the wheel load. As stated previously, this relationship, suggested by A. C. Benkelman and W. N. Carey, may provide a better index to the destructive effect of deflections caused by wheel loads.

Herner has made some pertinent comments from the viewpoint of someone primarily concerned with airport pavements. Figure 14 of the paper confirms his statement that deflections under standing loads are usually greater than those under moving wheel loads. Figures 9, 10, 11, 12, 15, and 16 confirm his suggestion that the amount of deflection measured by the G. E. Travel Gauge will vary depending upon the depth at which the reference rod is anchored. It is also true, however, that in many cases there is only a small difference between the deflections referenced at 8 feet compared to 18 feet. We also agree that the much lower rate of load repetition on airport pavements means that traffic effects are much less severe than for highway pavements so far as this factor is concerned.

It seems, however, that there is ample justification for being conservative when considering deflections during the design of a pavement. The report just issued on the WASHO test track (this closure was written on November 1, 1955) indicates that failures occurred with deflections not much above those suggested as limiting values by the California study. The WASHO track was less than 2 years old when testing was discontinued, and the net length of time under which traffic was operated was only about nine months. These factors seem to support the idea that present-day pavements subjected to constant repetition of load should not be expected to withstand much deflection if the pavement is to remain in first-class condition for a period of 10 years or more.

I agree completely with Herner's final observation that airport runways do not ordinarily present a serious problem, since load effects from planes are rather minor at high speeds, due both to the speed and to the fact that a considerable percentage of the load is airborne until the plane has reached the taxiway. It is widely known that taxiways are usually the first to show signs of distress, and this serves to point up the fact observed on highways that slow-moving heavy loads confined to the same wheel path are the most destructive of all, and it is under these conditions that deflections will be found to be of greatest magnitude.